PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
for Design-Build Ready Alternative Bid Package

for

WIDENING OF LITTLE BAPTISTE CREEK BRIDGES
Site Nos. 13X-0187/B1 & B2
Highway 401 – Station 19+600
Township of Tilbury, Chatham-Kent, Ontario
GWP 3034-19-00, WP 3230-19-01 & 3231-19-01
Assignment No. 3017-E-0006/0007
Work Item No. 07
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CONSULTING ENGINEERS

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G.W.P. 3034-19-00, Assignment No. 3017-E-0006/0007, Work Item No. 07

PART A – PRELIMINARY FOUNDATION INVESTIGATION PORTION OF THE REPORT

1. INTRODUCTION

The Ministry of Transportation Ontario (MTO) has retained WSP as the Prime Consultant, to provide services for the widening of EBL and WBL structures at three sites on Highway 401 under the request for proposal (RFP) for MTO Assignment No. 3017-E-0006/0007, Work Item No. 07. WSP retained Peto MacCallum Ltd. (PML) on behalf of MTO to provide foundation engineering services for this assignment. The Terms of Reference and Scope of Work for the Foundation Engineering services are outlined in the RFP for MTO Assignment No. 3017-E-0006/0007, Work Item No. 07.

This report is a Preliminary Foundation Investigation and Design Report for Design-Build Ready Alternative Bid Package for Little Baptiste Creek EBL and WBL Bridges located along Highway 401 at the crossing of Little Baptiste Creek in the Township of Tilbury, Chatham-Kent, Ontario. The subsurface investigation data was limited to available boreholes from a previously investigation supplemented by two (2) additional boreholes specifically drilled for the current assignment. Accordingly, further investigation will be required during detail design to establish or confirm / reassess the recommendations provided in this report.

2. SITE DESCRIPTION

Highway 401 in the area of the bridge site is elevated slightly above the natural topography, and accommodates two (2) lanes of vehicular traffic in each direction. The site is generally a flat area, with the exception of the highway embankments. The study area is surrounded by agricultural developments, and is located approximately 2.6 km east of the residential community of Tilbury.

3. FIELD INVESTIGATION PROCEDURES

The field work for the current foundation investigation involved drilling of two (2) boreholes to supplement the subsurface information from a previous investigation. The new boreholes are identified as LEB and LWB, located within the Highway 401 median, on the east and west side of Little Baptiste Creek, respectively. The boreholes were drilled to depths of 30 m below the existing ground surface. The locations, ground elevations and depths of drilling are summarized in Table 3.

Table 3: Borehole Location and Termination Depth

BOREHOLE NO.	LOCATION				DEPTH (m)	GROUND ELEVATION (m)
	NORTHING	EASTING	LATITUDE	LONGITUDE		
LWB	4 681 584.5	312 129.8	42.273794	-82.411139	30.0	178.3
LEB	4 681 593.3	312 162.2	42.273873	-82.410747	30.0	178.4

PML staff visited the site on August 17, 2019 to mark out the borehole locations. The appropriate utility companies cleared the underground services at the borehole locations. Public and private utility authorities were informed and all of the utility clearance documents were obtained prior to commencement of the drilling work.

PML staff used a portable GPS device to establish the borehole locations in the field. Subsequently, the locations and elevations of the drilled boreholes were surveyed by PML with a Sokkia SHC5000 Differential GPS unit equipped with a GCX3 (Network RTK rover) GNSS Receiver. The vertical and horizontal limits of accuracy of the Differential GPS unit are within 0.1 m and 0.5 m, respectively. All elevations reported in this report are referred to in MTM NAD 83 Northing and Easting (MTM Zone - ON11) Geodetic datum and expressed in metres.

The equipment used for drilling was owned and operated by London Soil Test Inc. (London Soil), located in London, Ontario. London Soil is a specialist drilling contractor and worked under the full-time supervision of a PML field supervisor. Boreholes LEB and LWB were drilled on between October 17 and 18, 2019. The boreholes were advanced using a D50-Turbo Track-mounted drilling rig equipped with 200 mm diameter hollow stem augers.

Refer to Drawings LBC-1 and LBC-2 in Appendix A for borehole location details.

Representative soil samples were recovered from the boreholes at 0.75 m intervals to a depth of 6.0 m and at 1.5 m intervals to a depth of 20 m, and at 3.0 m interval to the termination depth, using a conventional 51 mm OD split spoon sampler in accordance with the Standard Penetration Test (SPT) procedure. SPTs were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata. In addition, attempt was made to measure in-situ vane shear strength of clayey soil at depths where SPT values were below about 8 blows/300 mm, using a N-size (MTO) vane.

The groundwater conditions at the borehole locations were observed during the drilling operations by visual examination of the soil samples, sampler and drill rods as the samples were retrieved. In addition, water level measurements were taken in the open boreholes upon completion of drilling. A monitoring well, consisting of 50 mm outside diameter rigid PVC pipe, was installed adjacent to each borehole for groundwater level measurement. Water levels were measured using a Solinst flat tape water level reader.

Boreholes and monitoring wells were constructed and abandoned/decommissioned in conformance with the requirements of MTO guidelines and Ontario Regulation 903, amended by Ontario Regulation 372.

The Little Baptiste Creek water level was observed approximately at EL. 173.1 during the fieldwork carried out on October 17, 2019.

The recovered soil samples were delivered to PML's laboratory to conduct detailed visual examinations and index tests.

4. LABORATORY TEST PROCEDURES

Laboratory tests were conducted on representative SPT soil samples recovered during the fieldwork investigation work. Testing was conducted at PML's laboratory facility located in Toronto, Ontario. The laboratory testing program included the following:

- Natural moisture content determinations (43)
- Grain size distribution analyses (13)
- Atterberg limit tests (13)
- Consolidation test (1)

All the laboratory tests to determine soil index properties were performed in accordance with MTO test procedures, which follow the American Society for Testing Materials (ASTM) standards, with the exception of hydrometer tests (LS-702). The results of the grain size distribution analyses are presented in Figures GS-LC-1 to GS-LC-2. The results of the Atterberg Limit tests are presented in Figures PC-LC-1 to PC-LC-2. One-dimensional consolidation (ASTM D-2435) testing was conducted on one Shelby tube sample from borehole LEB and the results are presented in Figure L-1. All of the test results are summarized in the attached Record of Borehole Logs provided in Appendix A.

5. SITE GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Site Geology

In general, the project area is located within the St. Clair Clay Plains physiographic region. The Quaternary Geology map published by the Ontario Ministry of Northern Development and Mines (MNDM), indicates that the surface conditions in the area of the bridge site consist of Tavistock Till deposits; silty clay matrix. Based on the Bedrock Geology map (MRD126-REV1, 2011) published by the MNDM, the project area consists of Middle Devonian limestone, dolostone and shale of the Hamilton Group rock formation.

5.2 Previous Investigation

The field investigation for the existing bridges was carried out between December 16, 1958 and January 15, 1959, and consisted of five (5) boreholes drilled to depths ranging from 8.0 m (EL. 170.0) to 37.2 m (EL. 140.4) below the ground surface elevation at the time of investigation.

Based on the foundation investigation and design report (FIDR, Geocres No. 40J08-017), representative soil samples were recovered from the boreholes at frequent intervals to the termination depths of the boreholes, using a conventional 51 mm OD split spoon sampler, while simultaneously conducting SPTs to assess the strength characteristics of the substrata. In addition, 76 mm diameter thin wall tube (Shelby) undisturbed samples were also recovered to conduct complex laboratory tests. The laboratory tests consisted of index tests such as moisture content, Atterberg limits, and grain size distribution. The complex tests carried out on undisturbed samples involved one-dimensional consolidation testing and triaxial testing with pore pressure measurements.

Based on the previous investigation, the subsoil conditions in the area of the proposed structure is expected to consist of about 0.9 m to 1.7 m thick silty clay fill underlain by silty clay to the maximum borehole termination depth of 37.2 m (EL. 140.4). The upper most part of this clay layer to a depth of 7.0 m (EL. 171.1) to 8.0 m (EL. 169.0) appears to be desiccated and the SPT 'N'-values reported ranged from 9 blows to 27 blows, indicating stiff to very stiff consistency. Below these depths, the SPT 'N'-values reported ranged from 6 blows to 10 blows, indicating firm to stiff consistency.

Based on the report, groundwater was not encountered in any of the boreholes advanced and for the settlement analyses; it was assumed at the creek water level of EL. 175.0 observed during the investigation.

5.3 Current Investigation

The subsurface conditions encountered during the current investigation along with the field and laboratory test results are presented in the attached Record of Borehole Sheets. The borehole locations and stratigraphic profile sections are provided in Drawings LBC-1 and LBC-2. The boundaries between soil strata have been established at the borehole locations only. The boundaries of soil strata between and beyond the boreholes are assumed and may vary from location to location.

In general, the subsoil conditions immediately below the ground surface of the proposed structures consist of a layer of fill approximately 3.0 m comprising of clayey silt, with varying proportions of sand and gravel. The fill layer is underlain by a 27.0 m thick deposit of very stiff to soft silty clay to clayey silt till. Boreholes LEB and LWB were terminated in the firm clayey silt till at a depths of 30.0 m below the existing ground surface elevations. For classification purposes, the soils encountered at this site can be divided into two (2) distinct zones:

a) Clayey Silt, some sand, some gravel (Fill)
b) Silty Clay to Clayey Silt , Some Sand to Sandy, Trace Gravel (Till)

5.3.1 Clayey Silt, Some Sand, Some Gravel (Fill)

A layer of clayey silt fill was encountered just below the existing ground surface. The layer extends to depths of 3.0 m; elevations 175.4 and 175.3 below the existing ground surface in boreholes LEB and LWB, respectively.

The SPT 'N'-value recorded in this fill varied between 6 and 11 blows, indicating firm to stiff consistency. The moisture contents of samples tested from this fill were 17.4% and 28.7%.

5.3.2 Silty Clay to Clayey Silt, Some Sand, Trace Gravel (Till)

The fill in boreholes LEB and LWB was underlain by a deposit of silty clay to clayey silt till with varying proportions of sand and gravel. This till deposit was encountered at depths of 3.0 m (EL. 175.4 to 175.3) and it extended to the borehole termination depths of 30.0 m (EL. 148.4 to 48.3) below the existing ground surface. The SPT 'N'-values in this deposit generally ranged from 8 to 18 blows from EL. 175.4 to EL. 168.0, indicating stiff to very stiff consistency. Between EL. 168.0 and EL. 158.0, the SPT 'N'-values ranges from 2 to 7 blows. Within this depth, in-situ vane shear tests were carried out. The vane shear tests were performed at ten (10) locations between EL. 169.0 and EL. 156.5 within this till deposit and the uncorrected vane shear strengths (C_u) measured varied between 62 kPa and 95 kPa, with a sensitivity ratio value between 1 and 2, indicating stiff consistency, compared to soft to firm based on SPT 'N'-values. Below EL. 158.0, the SPT 'N'-values ranged between 10 and 14 blows, with the exception of one SPT 'N'-value of 6 blows in borehole LEB. The moisture contents of the samples tested from this till deposit varied between 15.5% and 29.0%.

The grain size distribution results of selected silty clay to clayey silt samples from this till deposit are provided in Figures GS-LC-1 and GS-LC-2, and the results of Atterberg limits for the same samples are provided in Figures PC-LC-1 and PC-LC-2 in Appendix A.

Sieve analysis tests were performed on thirteen (13) representative samples and the test results indicate that this deposit consists of 1 to 9% gravel, 14% to 27% sand, 37% to 44% silt, and 27% to 46% clay. Atterberg limit tests were performed on thirteen (13) representative samples and the test results indicate liquid limit values ranging from 27 to 46, plastic limit values ranging from 16 to 23, and corresponding plasticity index values ranging from 11 to 23. Based on the test results, the clayey soil may be classified as clay of low to medium plasticity (CL/CI), clayey silt/silty clay in the Unified Soil Classification System (USCS), or clayey silt to silty clay based on the MTO Soil Classification System.

One-dimensional consolidation testing was conducted on one Shelby tube sample obtained from borehole LEB considered to be representative of the site conditions. As part of the one-dimensional consolidation and particle size analysis of soils (LS-702), specific gravity testing was carried out on one sample from the silty clay till deposit. The specific gravity of the tested silty clay sample was 2.718. The bulk unit weight of the tested sample was 19.7 kN/m^3 with a corresponding dry unit weight of 15.5 kN/m^3.

The test results are provided in Appendix A. The following table summarizes the consolidation test results of the tested sample.

BOREHOLE NO./ DEPTH (m)	EFFECTIVE OVERBURDEN PRESSURE (kPa)	PRE-CONSOLIDATION PRESSURE (kPa)	OVER CONSOLIDATION RATIO (OCR)	INITIAL VOID RATIO (e_O)	COMPRESSION INDEX (C_C)
LEB/14.3	155	175	1.1	0.721	0.190

5.3.3 Groundwater

Groundwater was not encountered during and upon completion of drilling in boreholes LWB and LEB. The water level in the creek, which may ultimately control the groundwater level, was observed approximately at EL. 173.1 during the fieldwork.

A monitoring well consisting of 50 mm diameter PVC pipe was installed adjacent to boreholes LEB and LWB. Water level readings from the monitoring wells are summarized in Table 5.3.3.

Table 5.3.3: Water Level Readings in Monitoring Wells

MONITORING WELL (MW)	GROUND SURFACE ELEVATION (m)	TOP OF CASING ELEVATION (m)	MID-SCREEN DEPTH (m) (ELEVATION, (m)	WATER LEVEL MEASURED IN MONITORING WELL, DEPTH (m) (ELEVATION, m)		
				2019/10/24	2019/10/29	2019/11/07
LEB	178.4	179.2	6.9 (EL. 171.5)	Not Encountered	7.5 (EL. 170.9)	1.8 (EL. 176.6)
LWB	178.3	179.1	6.9 (EL. 171.4)	Not Encountered	7.5 (EL. 170.8)	1.8 (EL. 176.5)

Groundwater levels may fluctuate due to the influence of precipitation and seasonal change. The groundwater measurements were observed and measured prior to backfilling of the boreholes. Groundwater levels are provided in the Borehole Logs in Appendix A.

5.3.4 Soil Corrosivity

Three (3) representative soil samples were sent to SGS Canada Inc.'s (SGS) laboratory located in Toronto, Ontario, which is accredited by Canadian Analytical Laboratory Association (CALA). The corrosivity test results provided by SGS are presented in Appendix A. A summary of the test results is presented in the Table 5.3.4.

Table 5.3.4: Summary of Corrosivity Test Results

BOREHOLE ID	SAMPLE NO.	CORROSIVITY INDEX	SULPHIDE (%)	SOIL REDOX POTENTIAL (mV)	pH	RESISTIVITY (Ohm-cm)	CONDUCTIVITY (µS/cm)	SULPHATE (µg/g)	CHLORIDE (µg/g)
LEB	4	6.5	0.02	177	7.75	2260	443	78	170
LEB	11	4.5	0.45	134	8.42	3520	284	110	140
LWB	10	4.5	0.38	106	8.24	3770	265	130	45

The Foundation Investigation portion of the report was prepared by Mr. K. Amatya, P.Eng. and Mr. N. Rahman P.Eng, Project Engineers. and reviewed by Mr. G. Uwimana, M.Eng., P.Eng, Senior Engineer. Mr. R. Ng, MBA, PhD, P.Eng. Principal Consultant conducted an independent review of the report.

Yours very truly

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PART B – PRELIMINARY FOUNDATION DESIGN PORTION OF THE REPORT

6. PROJECT DESCRIPTION

6.1 General

The Ministry of Transportation Ontario has proposed to widen the existing Highway 401 from four lanes to six lanes (widening 1 lane in each direction), from east of Essex Road 42 easterly to west of Merlin Road. The proposed widening will be along the existing median between Highway 401 EBL and WBL with Ontario Tall Wall concrete Median barrier. The Ministry requires a Design-Build Ready alternative package for the delivery of this project.

This report provides recommendations for foundation design based on interpretation of the previous report and on the geotechnical data presented in the factual portion for the new investigation presented in this report (Part A) and the details provided on the General Arrangement (GA) drawings for the proposed eastbound and westbound bridges on Highway 401 at the crossing of Little Baptiste Creek in the Town of Tilbury, Ontario. Based on the GA drawings, it is proposed to widen the existing bridges to the median with a single-span structure similar to the existing structures.

The discussions and recommendations presented in this report are based on the information provided by WSP and the factual data obtained during the geotechnical investigation carried out by PML.

This preliminary foundation investigation and design report with the interpretation and recommendations are intended for the use of WSP on behalf of MTO. Any other parties including the Project CO Team or design-build contractor may use the information presented in this report at their own risks.. Where comments are made on construction, they are provided only to highlight those aspects, which could affect the design of the project. Contractors must make their own interpretation of the factual information provided in Part A of the report, as it may affect equipment selection, proposed construction methods and scheduling.

6.2 Existing Structures

Based on the General Layout Drawing D 4382-1 dated July 1959 and information supplied by WSP dated November 15, 2019 via email, the existing bridges on EBL and WBL are rigid frame structures with a clear span of 10.7 m, and supported on approximately 1.676 m wide strip footings placed at about EL. 173.1. The footings were designed to carry an allowable bearing pressure of approximately 145 kPa. Both bridges have different approach slab lengths ranging from 5.3 m to 7.5 m.

The abutment walls appear to be in good condition with several noticeable cracks running from the top of the wall to the exposed ground surface along the bank. The cracks were observed to be narrower at the top compared to near the ground surface, indicating that the cracks may have been the result of differential settlements, which may have ceased over the past sixty (60) years since the construction of the bridges. During the previous site visit conducted by PML on October 20, 2013, the slopes on both sides of the abutments were heavily vegetated. The embankment slopes were observed to be in stable condition and neither erosion nor scour were noted/observed around the toes of the embankments or abutments.

6.3 Proposed Structure

Based on the GA drawings dated October 2019 that were provided by WSP, the proposed structure widening will have a clear span of 10.7 m, which is the same as for the existing bridges. It is proposed to support the widened portion of the new abutments on piles. The pile cap is shown to be founded at EL. 173.1, the same as the existing strip footings. The design grade of the approaches at the east and west abutments will be approximately EL. 178.5.

7. FOUNDATION RECOMMENDATIONS

7.1 Subsoil Conditions

Summaries of the subsoil and groundwater conditions are provided in Part A of this report.

7.2 Foundation Alternatives

The foundation alternatives discussed below are for preliminary design purposes to facilitate development of conceptual design criteria for the Design-Build Ready project delivery alternative package. Additional foundation investigation and design services would be required to produce a detail design level Foundation Design Report.

For comparison purposes, the following Table 7.2 provides the advantages, disadvantages, risks and consequences of the foundation alternatives to support the proposed structure.

Table 7.2 – Comparison of Foundation Options

FOUNDATION TYPE	ADVANTAGES	DISADVANTAGES	RISKS / CONSEQUENCES	RELATIVE COST
Driven Steel Piles	Higher confidence level in settlement performance, which is the critical issue for this bridge widening May not require deep excavations for forming pile caps	Potential vibration induced during driving Potential requirement to design for corrosion protection Closed-end pipe pile is considered as displacement pile, which will impact the existing footings during installation. Hence, this type of pile is not considered feasible and practical at this site	Steel piles may require corrosion protection, in which case the corrosion protection would need to be designed by specialists.	Moderate
Caissons	Considering in-situ shear strengths measured during current investigation and existing groundwater conditions, caissons are not considered feasible at this site location due to the high cost and limited resistance available due to the subsoil conditions		Higher risk of undermining existing shallow foundations due to soil removal in caisson installations	Relatively high
Shallow Foundations	Same construction as existing, but now under different circumstances considering the depth that would be required for construction at the same founding level and the importance of acceptable differential settlement performance for widenings	Deep excavations and roadway protection/shoring in the order of 9 m would be required to construct spread footings for widenings at the same elevation as existing spread footings Longitudinal slip joints would be required to accommodate the differential settlements between the spread footing foundations for the existing structure and the new widenings	Higher risk of excessive settlement and resulting distresses to bridges due to differential settlements	Foundation cost relatively low, but more cost uncertainty due to deeper excavation and shoring requirements

Based on the evaluation as summarized in Table 7.2, driven H-piles are the preferred foundation alternative for the proposed widenings from a foundations engineering perspective.

7.2.1 Driven Steel Piles

The proposed structure may be supported on steel H-piles or open-ended pipe piles (if displacement concerns are adequately addressed by the design-build entity) driven to the tip elevations indicated on Table 7.2.1a, and terminated in the cohesive till deposit. The geotechnical resistances provided below are primarily derived from shaft friction. As a result, the spacing between piles should be at least three times the diameter or width of the piles. The removal of soil inside the open-ended steel tube pile may be practically limited to a depth that is structurally required for placing reinforcement and concrete to connect the pile to the pile cap. Further, consideration may be given to locating the pile caps supporting the abutments in a pile bent configuration or at an elevation that is structurally feasible and below the frost penetration depth of 1.0 m, to avoid excessively deep excavations and shoring requirements adjacent to existing footings.

It is anticipated that driving of piles will commence from the proposed founding elevation of the pile cap, adjacent to the existing footings. In order to prevent damages to the existing structures, the piles should be lowered in approximately 400 mm to 500 mm diameter pre-augured holes extending to a depth of 3.0 m below the founding levels of the existing footings, i.e., to about El. 170.0, and driven to the tip elevations suggested in Table 7.2.1a.

Construction of the deep foundation should conform to OPSS.PROV 903, amended by SP 109F57.

Table 7.2.1a summarizes the geotechnical resistances of open-ended pipe and steel H piles for the preliminary design purpose.

Table 7.2.1a: Geotechnical Resistances for Preliminary Design

TYPE OF PILES	EMBEDMENT LENGTH BELOW EXISTING FOUNDATION DEPTH (m)	PILE TIP ELEVATION (m)	FACTORED AXIAL RESISTANCES (kN)	
			AT ULS	AT SLS
324 mm O.D, diameter, 6.3 mm thick Open-ended Pipe Pile	15	155.0	260	200
	18	152.0	310	240

Table 7.2.1a: Geotechnical Resistances for Preliminary Design

TYPE OF PILES	EMBEDMENT LENGTH BELOW EXISTING FOUNDATION DEPTH (m)	PILE TIP ELEVATION (m)	FACTORED AXIAL RESISTANCES (kN)	
			AT ULS	AT SLS
HP 310 x 110 Steel H-Piles	15	155.0	310	240
	18	152.0	390	300
HP 310 x 79 Steel H-Piles	15	155.0	290	210
	18	152.0	360	270

It is estimated that settlement of individual piles (SLS condition) may be less than or equal to 10 mm for the factored axial resistance at Serviceability Limit State (SLS) provided in Table 7.2.1a.

The lateral resistance of the piles may be computed using the equation provided below for cohesive soils, and the soil parameters recommended in Table 7.2.1b.

a) Cohesive Soils (Davison, 1970)

$k_s = 67\ \tau_u/d$

where τ_u = Undrained shear strength

d= Pile diameter or width, m

Table 7.2.1b: Parameters for Calculation of Coefficient of Lateral Subgrade Reaction

SOIL BOUNDARY ELEVATION		SOIL TYPE	UNDRAINED SHEAR STRENGTH (kPa)	n_h Values (kN/m^3)
FROM	TO			
170.0	166.0	Very stiff to stiff silty clay to clayey silt	100	-
166.0	155.1	Stiff silty clay to clayey silt	60	-

The ultimate lateral resistance may have to be reduced, based on pile spacing for open-ended pipe piles. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 7.2.1c:

Table 7.2.1c – Pile Spacing and Group Reduction Factor[1]

CONDITION	PILE SPACING (CENTRE TO CENTRE)	GROUP REDUCTION FACTOR
Pile group oriented perpendicular to the direction of loading	≥ 2.5D	1.0
Pile group oriented parallel to direction of loading	8 D	1.0
	6 D	0.7
	4 D	0.4
	3 D	0.25

1. Terzaghi, K., Peck, R.B. & Mesri, G. (1996). *Soil Mechanics in Engineering Practice* (3rd ed.). New York, NY: John Wiley & Sons, Inc.

Boulders and cobbles were not encountered during the current investigation. The piles may not require pile tip reinforcement during driving through the existing soil overburden. However, if pile reinforcement is considered during construction, oversized driving shoes similar to Ontario Provincial Standard Design (OPSD) 3000.100 (Foundation Piles Steel H-Pile Driving Shoe) or Titus H bearing pile point are not suggested. These types of pile tip reinforcement may reduce the shaft friction and may lead to overrun, especially when the pile capacity is derived from shaft friction.

7.2.2 Shallow Foundation

The existing structures are supported on shallow footings placed at about EL. 173.1. The existing ground level in the area of the median slopes from the road level (EL. 178.5) with a forward slope of 2H:1V towards the creek. Construction of footings for the proposed abutments may require about 4.0 m to 4.5 m deep excavation from the existing forward slope and related roadway protection/shoring. Settlement monitoring would be required to measure differential settlements between the existing bridge footings and the new footings for the widened portions of the bridges and to monitoring existing structures adjacent to foundation excavations during construction. The settlement monitoring program would be required for a specified period of time during the construction phase and upon completion of construction. Baseline readings would be taken prior to commencement of the construction work for the widening to monitor movement of existing structures during the construction phase and immediately upon completion of construction to document differential settlements. Subsequent changes from the baseline readings would be monitored in conjunction with pre-established criteria for dealing with Cautionary, Review, and Alert level settlement/movement readings for safety purposes. Such a monitoring program could be developed, by MTO and WSP, if this foundation option is selected.

The proposed east and west abutments could be supported on footings placed approximately at EL. 173.1. The geotechnical resistances provided on Table 7.2.2 for a 2.1 m wide footing may be considered for the design of the proposed widening bridges.

Table 7.2.2 – Founding Elevation and Geotechnical Resistance for Shallow Foundation

LOCATION	FOUNDING ELEVATION	FACTORED GEOTECHNICAL RESISTANCE AT ULS (kPa)	GEOTECHNICAL RESISTANCE AT SLS (kPa)
East Abutment	173.1	240	160
West Abutment			

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.10.4 of the CHBDC (2014). The previous foundation report from 1959 predicted settlements in the order of 165 mm of settlements under the projected loadings. Depending on the degree of consolidation of the founding soil under the highway embankment at the location of the proposed foundations for the widenings, the total settlement of new footings under the recommended SLS loads recommended may be in the order of 100 mm and 120 mm.

A modulus of subgrade reaction of 12,000 kN/m^3 for the soil at the founding level (EL. 173.1) of footing may be assumed for the design purposes.

The existing footings have been in place for almost sixty (60) years; long enough for cessation of time-related settlements. Accordingly, differential settlements are expected between the new and existing structures. In view of this, it is advisable to provide a “slip or Isolation” joint between the existing and the new structures to accommodate the differential settlements.

The sliding resistance of footings against lateral loads between the concrete footing and subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). For cast-in-place concrete footings constructed on concrete working slabs and on top of very stiff clayey silt, the following friction factors (tanδ) are suggested:

- Cast-In-Place footing on concrete working slab: = 0.6
- Cast-In-Place concrete working slab on very stiff clayey silt: = 0.4

7.2.3 Rehabilitation of Existing Bridges

For the purpose of bridge rehabilitation (without addition of new median load) and evaluation of existing footings, the previous assessment in Section 6.2 of the previous PML report, based on previous codes, has been re-evaluated. Based on current practice and the 2014 CHBDC requirements, we suggest that the design values be updated to factored geotechnical resistances of 200 kPa at SLS and 375 kPa at ULS.

7.2.4 Impact on Existing Footings

The structural arrangement, orientation, size, and spacing of shallow foundations placed close/adjacent to each other should be designed to minimize the degree of overlap of the foundation soil pressure bulbs and/or ensure that the soil pressures within the pressure bulb overlap zones do not exceed the design/specified geotechnical resistance values.

The influence factors in the bearing capacity calculation/assessment include footing configurations, soil compressibility, and loading. Stuart (1962)[2], Mandel (1965)[3], and West and Stuart (1965)[4] considered that the influence of adjacent footings for soils with low angles of internal resistance, similar to the subgrade soils at this site, may not be significant considering that the interaction effects are reduced as the length (L) to width (B) ratio (L/B) exceeds one and the compressibility of soil may have a lessening effect on the interference. The risk of punching shear failure is considered negligible based on assessment of the foundation investigation and design data presented in this report. Although, in a qualitative sense, the interference effect may not be significant in the bearing capacity calculations, the design and construction methodology shall be devised and carried out in manner that limits further impact on the existing footing(s).

The footings for the proposed bridge are expected to be placed along the longitudinal axes of the existing footings with the length of each new footing running parallel to the longitudinal axes of the existing footings. The new footings are to be founded on clayey soil. The width (B) of each new strip footing is assumed to be about 2.0 m with an approximate length (L) of 12.6 m. Assuming a footing length to width ratio (L/B) of 6.3, the effect of the new footings on the existing footings is considered to be minor. Factored geotechnical resistances in the order of 200 kPa at SLS and 375 kPa at ULS may be assumed for checking the existing footings for the current load and additional load from the abutment backfill and the new footing. The final configuration and design should be reviewed and the conditions analysed and examined by a geotechnical engineer to ensure that the recommendations and adopted soil parameters are appropriately incorporated.

7.3 Approach Embankments

Based on the GA drawing, the existing ground elevation of the east and west approaches is at about EL. 178.5. The height of the fill required in the median approaches to match the existing road elevation, is about 0.2 m to 0.3 m.

Based on the required fill, no instability and settlement problems are anticipated. The fill should consist of suitable fill material compacted in conformance with OPSS 501. Any spongy or soft area observed within the base of the excavation should be removed before placing the fill.

2. Stuart, J.G. (1962). Interference between foundations with special reference to surface footings in sand, *Geotechnique* 12(1),15-22.
3. Mandel, J. (1965). Interférence platique de semelles filantes; *Proceedings, Sixth International Conference on Soil Mechanics and Foundation Engineering,* Montreal, Vol II, 127-131.
4. West, J.M. & Stuart, J.G. (1965). Oblique loading resulting from interference between surface footings on sand, *Proceedings, Sixth International Conference on Soil Mechanics and Foundation Engineering,* Montreal, Vol II, 214-217.

7.4 Lateral Earth Pressures

Earth retaining walls or abutments should be designed to resist the horizontal earth pressure imposed by the backfill and any surcharge load. The earth pressure for concrete structures should be computed as per Clause 6.12.2 of Canadian Highway Bridge Design Code (CHBDC, 2014). The lateral earth pressure, p (kPa), may be computed the following equation, assuming a triangular pressure distribution:

$$p = K(\gamma h_1 + \gamma' h_2 + q) + \gamma_w h_2 + C_p + C_s$$

where K = Coefficient of lateral earth pressure (dimensionless)

γ = Unit weight of backfill material above assumed water level (kN/m^3)

γ' = Unit weight of submerged backfill (γ_{sat} - γ_w) material below assumed water level (kN/m^3)

γ_w = Unit weight of water (9.8 kN/m^3)

h_1 = Depth below final grade above design water level (m)

h_2 = Depth below design water level (m)

q = Surcharge load (kPa)

C_p = Compaction pressure (kPa) (Clause 6.12.3 of CHBDC, 2014)

C_s = Earth pressure from seismic events, (kPa) (Clause 4.6.5 of CHBDC, 2014)

Ontario Provincial Standard Specifications (OPSS.PROV 1010) Granular 'A' or 'B Type II' should be used as backfill material behind the wall and compacted in accordance with the requirements specified in the OPSS 902 (Excavation and Backfilling of Structures), amended by SP 109S12. The backfill material should be placed in layers not exceeding 200 mm (8 in.) in thickness before compaction.

Heavy vibratory compaction equipment adjacent to retaining structures should be restricted to limit the compaction pressure described in Clause 6.12.3 of the CHBDC, 2014. Restrictions on compaction near the retaining wall shall be as specified in OPSS 902, amended by SP 109S12. The type of compaction equipment and the compaction procedure that can be used for this purpose should be in accordance with OPSS.PROV 501 (Construction Specification for Compacting). Table 7.4 provides the recommended earth pressure coefficients.

Table 7.4 – Earth Pressure Coefficients

PARAMETERS	OPSS GRANULAR 'A'	OPSS GRANULAR 'B' TYPE II	FILL	CLAYEY SILT TO SILT CLAY
Internal Friction Angle, (degrees)	35°	30°	Effective Stress Value 24°	Effective Stress Value 20°
Unit weight, γ (kN/m^3)	22.5± 0.3	21.5 ± 0.3	18.0 ± 0.5	19.5 ± 0.5
Coefficient of Active Earth Pressure, Ka	0.27	0.33	0.42	0.49
Coefficient of Earth Pressure at Rest, Ko	0.43	0.5	0.59	0.65
Coefficient of Passive Earth Pressure, Kp	3.69	3	2.37	2.04

The coefficient of earth pressure "at rest" should be used for design of rigid and unyielding walls where sufficient movement of the structure wall is not permitted. For unrestrained structures, the active earth pressure coefficient should be employed.

A weeping tile system (OPSS 405 and OPSD 3190.100) and/or weep holes should be installed to minimize the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or geotextile (FOS 125 µm to 250 µm) to prevent migration of fines into the system. The drainage pipe should be installed on a positive grade and lead to a frost-free outlet. The geotextile should conform to OPSS.PROV 1860.

Backfilling adjacent to retaining structures should be carried out in conformance with OPSS 902, amended by SP 109S12. The minimum requirement of backfill material should be in accordance with OPSD 3101.150 for abutment and for retaining walls, it should be in accordance with OPSD 3121.150.

7.5 Seismic Considerations

The Spectral (Sa(T), where T is in seconds) and Peak Ground Acceleration (PGA) for the project site is 0.110 (Sa (0.2)) and 0.067 (2%/50 years), respectively, based on the longitude and latitude coordinates of the proposed structure (National Building Code of Canada, 2015). The PGA_{ref} for the site is 0.054 in accordance with Clause 4.4.3.3, CHBDC (2014). The soil below the founding level at this site for seismic design purposes is classified as Site Class D in accordance with Clause 4.4.3.2, CHBDC 2014.

In accordance with Clause 4.4.4, CHBDC (2014), a seismic performance category of 1 (major–route and other bridges) is considered for the site. No seismic related design considerations are anticipated for this site.

7.6 Scour Protection

Cognizant of the potential harmful effects of stream flow to foundation systems carrying bridges in flood plains and near flowing waterbodies such as river, creek, stream and channel, scour and erosion protection is possibly required. Assessment and evaluation to determine if bridge scour protection will required for this site may be carried out during the detail design stage.

The assessment, analysis, design, nature and extent of the bridge scour protection that may be required at this site is the responsibility of a qualified hydraulic engineer experienced in the field. It is suggested that the bridge scour and stability analyses be carried out in accordance with the guidelines set out in the Hydraulic Engineering Circular (HEC) Nos. 18 (Evaluating Scour at Bridges), 20 (Stream Stability at Highway Structures) and 23 (Bridge Scour and Stream Instability Countermeasures), and CHBDC (2014).

It is anticipated that scour protection may be required on creek banks and adjacent to the abutments. Rock protection or riprap may be provided to a minimum height of 1.0 m above the high flood level expected in the creek to the toe of the slope and into the creek bed within the plan limits of the bridges. The construction for rock protection or rip-rap should be in accordance with OPSS.PROV 511.

7.7 Frost Protection

All pile caps or footings shall be provided with a minimum of 1.0 m of earth cover or equivalent thermal insulation as protection against frost action as per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario).

8. ROADWAY PROTECTION

For the construction of the proposed structure, it may require a properly designed temporary roadway protection system. The earth pressure values presented in Table 7.4 may be used for design. Temporary roadway protection shall be designed to meet at least a Performance Level of 2 and constructed in accordance with OPSS.PROV 539 (Construction Specification for Temporary Protection Systems), amended by SP 105S09. The Contractor shall be responsible for the selection, detailed design and performance of the roadway protection system. OPSS.PROV 539, amended by SP 105S09, also calls for monitoring of the roadway protection system by the Contractor to check the horizontal and vertical displacements of the roadway.

9. EXCAVATION

All excavations should be carried out in accordance with the Occupational Health and Safety Act (OHSA) and MTO Regulations for Construction Projects. According to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria, the existing fill is classified as Type 3 soils. The very stiff cohesive soil is considered as Type 2 soil. The stiff cohesive soil is considered as Type 3 soil. Soils below groundwater table and soils showing persistent seepages are considered having the characteristics of a Type 4 soil. The open cut procedure will be governed by soils with the highest soil type number.

The protection system for excavations should be in accordance with OPSS.PROV 539, amended by SP 105S09. Construction Specifications for Excavating and Backfilling–Structures should be in accordance with OPSS 902, amended by SP 109S12. All excavated surfaces should be kept free of frost and water during the period of construction. Runoff shall be directed away from open excavations and should not be allowed to flow into the excavation. Excavated material shall not be stockpiled on top of the excavation.

Prior to excavation, the locations and depths of existing underground utilities should be verified. All underground utilities that might be exposed and become unsupported as a result of the excavation should be properly supported to avoid potential damage.

Based on GA drawing, the pile cap is to be founded at EL. 173.1, about 5.5 m to 6.0 m below the existing ground surface. The protection system for excavation will be required in accordance with OPSS.PROV 539, amended by SP 105S09. A shoring system consisting of sheet piles or of H-piles with timber lagging may be used for excavation.

Alternatively, the pile cap may be founded at as high an elevation as possible but below the frost depth.

The base of the pile cap excavation should be protected from disturbance by placing a minimum 100 mm thick lean concrete, following the removal of existing fill material.

10. CONSTRUCTION CONSIDERATIONS

The "red flag" issues outlined in the following subsections and the recommended methods of overcoming these issues are intended to alert and aid the designer and the contractor. These comments and recommendations are based on the conditions revealed during the investigations and no responsibility is assumed by the consultants or the Client for alerting the contractor to all critical issues for each foundation alternative. The requirements to deliver acceptable quality of construction remain the responsibility of the contractor.

10.1 Groundwater Control

It is anticipated that up to 6.0 m of excavations will be required to found the proposed pile caps at both abutments at approximately EL. 173.1, which is approximately the same as the creek level and is below the prevailing groundwater approximately at EL. 176.5. A temporary protection system (i.e. cofferdam) will be required for dewatering operations to permit construction in the dry. A cofferdam consisting of sheet piles or H-piles with timber lagging may be used for excavation and dewatering. Dewatering may be carried out from the sump pumps located along the periphery of the cofferdam.

Alternatively, the pile cap could be founded at about 2.5 m below ground surface EL. 176.0. With this option, conventional sump pumping techniques are considered to be adequate to mitigate any surface runoff and seepage from localized soil fissures at the excavation depth

In any case, groundwater should be lowered a minimum of 0.5 m below excavations for construction in-the-dry. The Contractor is responsible for the selection, design and performance of the groundwater control measures.

The contractor shall be responsible for the selection, performance and detailed design of the shoring and dewatering system including cofferdam. The dewatering system should be designed to conform to the requirement of OPSS.PROV 517, SP 517F01 and NSSP FOUN0003.

In accordance with SP 517F01, the dewatering system should be designed by a designer with a minimum 5 years of experience in the field. A preconstruction survey is not required due to the relatively shallow depth of dewatering and the relatively large distances to critical private properties.

10.2 Soil Corrosivity

A total of three (3) samples from the fill and clayey silt to silty clay deposit were tested for soil corrosivity and potential exposure of concrete to sulphate attack. A summary of the results of chemical analyses are provided in section 5.3.4 of Part A of this report. The sulphate concentration varied from 78 µg/g to 130 µg/g (0.0078% to 0.013%). Compared to the values suggested in Canadian Standard A23.1-14, the effect of fill material on buried concrete is considered negligible. The chloride contents of the samples from the fill ranged from as low as 45 µg/g to 170 µg/g (0.0045% to 0.017%). Generally, the concentration value in excess of 250 ppm (0.025%) leads to corrosive environment for buried metals or reinforcing steel. The potential for corrosive environment of this fill is assessed to be low to moderate.

Electrical resistivity less than 2000 ohm-cm generally leads to highly corrosive environment for steel elements in contact with soil. The resistivity values of samples ranged from 2260 ohm-cm to 3770 ohm-cm. The test results suggest that a corrosive environment exists at this site for steel elements in contact with fill where the resistivity was less than 2000 ohm-cm. The pH values of fill samples ranged from 7.75 to 8.42.

Generally, it may be advisable to use imported backfill material selected to provide a more benign chemical environment for the approach embankments. Otherwise, measures to mitigate the impact of the chemical environment could be considered.

11. CLOSURE

The Foundation Design portion of the report was prepared by Mr. N. Rahman, P.Eng. and reviewed by Mr. G. Uwimana, M.Eng., P.Eng, Senior Engineer. Mr. R. Ng, MBA, PhD, P.Eng. Principal Consultant conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

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APPENDIX A

Borehole Locations Plan and Soil Strata Drawings LBC-1 and LBC-2

Explanation of Terms Used in Report

Record of Borehole Sheets

Results of Grain Size Distribution Analyses – Figures GS-LC-1 to GS-LC-2

Results of Atterberg Limit Tests – Figures PC-LC-1 to PC-LC-2

Consolidation Test Results Figures L-1

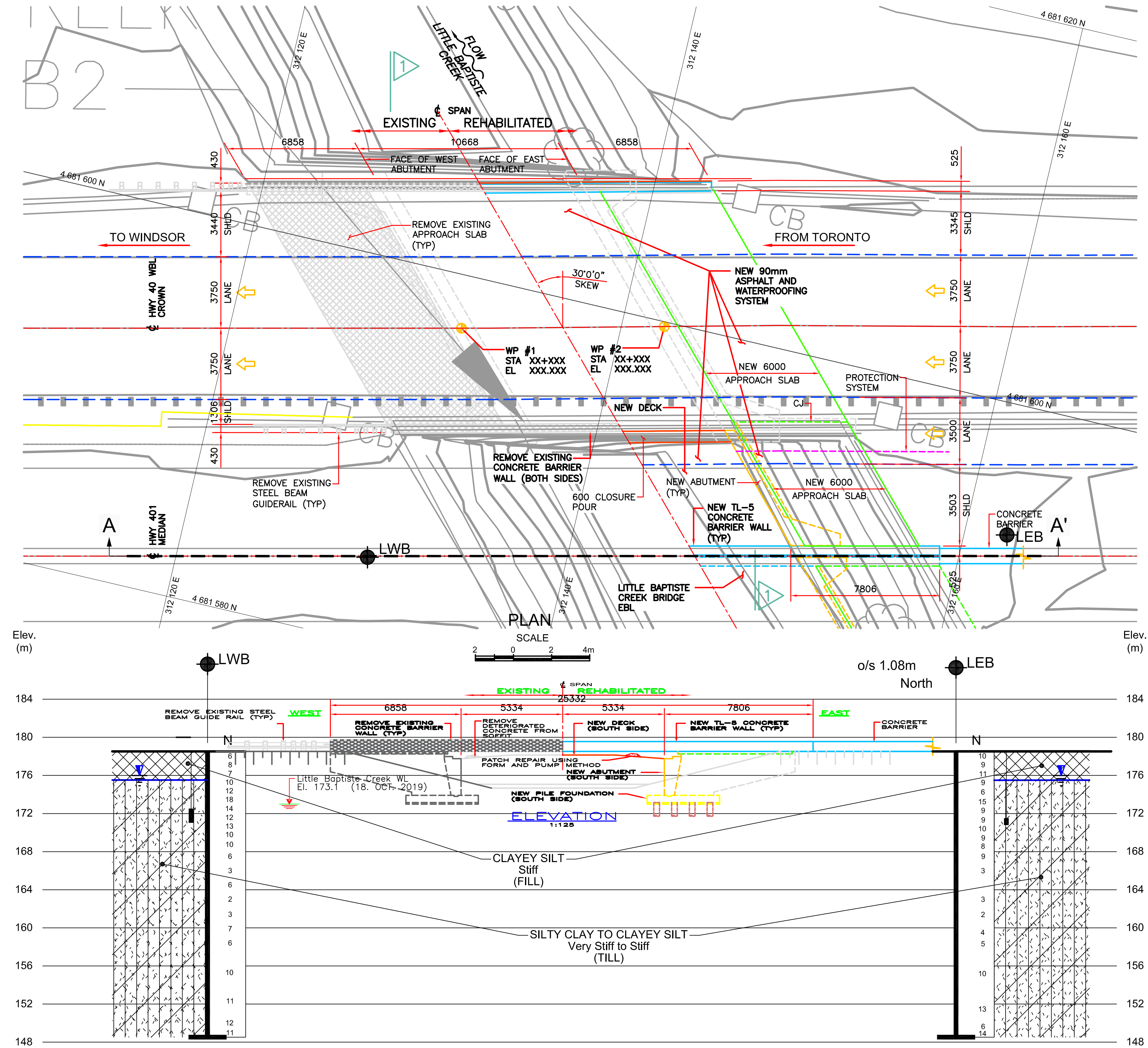
Results of Chemical Tests provided by SGS Canada Inc.

CONT No
Work Item No. 7
GWP No 3034-19-00

HIGHWAY 401 WIDENING
LITTLE BAPTISTE CREEK BRIDGE – WESTBOUND
BOREHOLE LOCATION PLAN AND SOIL STRATA

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS



PROFILE ALONG A-A'

SCALE

HORIZONTAL 2 0 2 4m

VERTICAL 5 0 5 10m

KEY PLAN

1km 0 1km 2km

LEGEND

- Borehole Location
- N — Blows/0.3m (Std. Pen Test, 475 J/blow)
- Monitoring Well
- Water Level in Monitoring Well (November 2019)

BH No	ELEVATION	NORTHINGS	EASTINGS
LWB	178.3	4 681 584.5	312 129.8
LEB	178.4	4 681 593.3	312 162.2

– NOTE –

The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

NOTES:

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
2. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
3. DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.

Reference WSP Ltd. Drawing: 18M-02111-07-302-001GA.dwg, dated October 2019.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 40J8-73

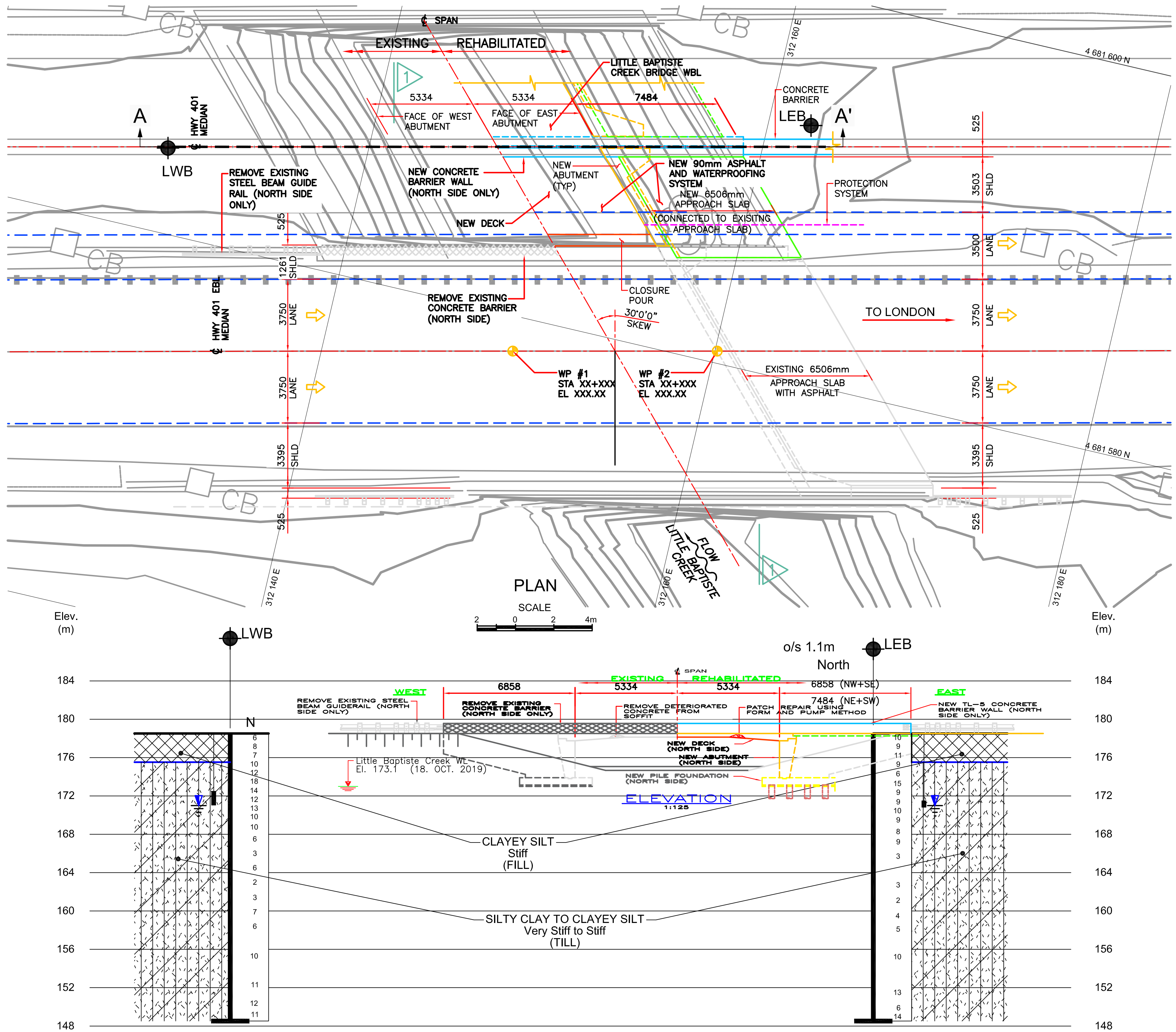
HWY No 401WBL			DIST WEST REGION
SUBM'D NL	CHECKED KA	DATE NOV. 08, 2019	SITE 13-187/1
DRAWN NL/MM	CHECKED NR	APPROVED RN	DWG LBC-1

CONT No
Work Item No. 7
GWP No 3034-19-00

SHEET

HIGHWAY 401 WIDENING
LITTLE BAPTISTE CREEK BRIDGE – EASTBOUND
BOREHOLE LOCATION PLAN AND SOIL STRATA

Peto MacCallum Ltd.
CONSULTING ENGINEERS



PROFILE ALONG A-A'

SCALE

HORIZONTAL 2 0 2 4m

VERTICAL 5 0 5 10m

KEY PLAN

LEGEND

- Borehole Location
- N — Blows/0.3m (Std. Pen Test, 475 J/blow)
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- Water Level in Monitoring Well (October 2019)

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Reference WSP Ltd. Drawing: 18M-02111-07-303-001GA.dwg, dated October 2019.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 40J8–73

HWY No 401 EBL			DIST WEST REGION
SUBM'D NL	CHECKED KA	DATE DEC. 10, 2019	SITE 13–187/2
DRAWN NL/MM	CHECKED NR	APPROVED RN	DWG LBC–2

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

COMPOSITION: SECONDARY SOIL COMPONENTS ARE DESCRIBED ON THE BASIS OF PERCENTAGE BY MASS OF THE WHOLE SAMPLE AS FOLLOWS:

PERCENT BY MASS	0 – 10	10 – 20	20 – 30	30 – 40	> 40
	TRACE	*SOME*	*WITH*	*ADJECTIVE (SILTY)*	*AND (AND SILT)*

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE
F V	FIELD VANE		

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
DTPL		DRIER THAN PLASTIC LIMIT
APL		ABOUT PLASTIC LIMIT
WTPL		WETTER THAN PLASTIC LIMIT
e_{max}	1,%	VOID RATIO IN LOOSEST STATE
e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No LWB

METRIC

G.W.P. 3034-19-00	LOCATION Coords: 4 681 584.5 N; 312 129.8 E		ORIGINATED BY M.M.
DIST West Region HWY 401	BOREHOLE TYPE Continuous Flight Hollow Stem Augers		COMPILED BY K.A.
DATUM Geodetic	DATE 2019.10.17	LATITUDE 42.273794 LONGITUDE -82.411139	CHECKED BY R.N.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
178.3	Ground
0.0	CLAYEY SILT some sand, trace gravel Stiff, Brown, Moist (FILL)
174.6 / 3.7	CLAYEY SILT, some sand, trace gravel Very stiff to stiff, Grey, Moist (TILL)
163.3	

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	6	
2	SS	8	
3	SS	7	
4	SS	10	
5	SS	12	
6	SS	18	1 18 42 39
7	SS	14	
8	SS	12	
9	SS	13	2 16 43 39
10	SS	10	
11	SS	10	
12	SS	6	
13	SS	3	1 19 39 41
	VANE		
14	SS	6	

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165, 164); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

Field vane at approx. elev. 164.7: +1

Continued Next Page

+3, ×3: Numbers refer to Sensitivity ○3% STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No LWB

METRIC

G.W.P. 3034-19-00	LOCATION Coords: 4 681 584.5 N; 312 129.8 E		ORIGINATED BY M.M.
DIST West Region HWY 401	BOREHOLE TYPE Continuous Flight Hollow Stem Augers		COMPILED BY K.A.
DATUM Geodetic	DATE 2019.10.17	LATITUDE 42.273794 LONGITUDE -82.411139	CHECKED BY R.N.

SOIL PROFILE: ELEV / DEPTH, DESCRIPTION, STRAT PLOT | SAMPLES: NUMBER, TYPE, "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	Field Vane (kPa) / Sensitivity	GR	SA	SI	CL
163.3 15.0	(Cont'd) CLAYEY SILT, some sand, trace gravel Stiff, Grey, Moist (TILL)	15	SS	2					
			VANE		$+^2$				
		16	SS	3		2	17	41	40
		17	SS	7					
			VANE		$+^1$				
		18	SS	6					
			VANE		$+^2$				
		19	SS	10		9	27	37	27
		20	SS	11					
		21	SS	12		2	19	41	38
148.3		22	SS	11					

Elevation scale: 163, 162, 161, 160, 159, 158, 157, 156, 155, 154, 153, 152, 151, 150, 149

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MTO 19KF030A - WO7.GPJ ONTARIO MTO.GDT 12/11/19

RECORD OF BOREHOLE No LWB

METRIC

G.W.P. 3034-19-00 LOCATION Coords: 4 681 584.5 N; 312 129.8 E ORIGINATED BY M.M.
DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
DATUM Geodetic DATE 2019.10.17 LATITUDE 42.273794 LONGITUDE -82.411139 CHECKED BY R.N.

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
148.3 30.0	End of borehole										

NOTES:
1. Groundwater was not encountered during or upon completion of drilling.
2. No cave-in was noted upon extraction of augers.

▼ Groundwater level measured in monitoring well

Monitoring Well Readings:

Date	Depth (m)	Elev.
Oct.24/'19	Dry	-
Oct.29/'19	7.5	170.8
Nov.07/'19	1.75	176.6

Monitoring Well Legend:
- Stick-up Monument
- Bentonite
- Filter Sand
- 19 mm PVC Screen
- Filter Bottom

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No LEB

METRIC

G.W.P. 3034-19-00 | LOCATION Coords: 4 681 593.3 N; 312 162.2 E | ORIGINATED BY M.M.
DIST West Region HWY 401 | BOREHOLE TYPE Continuous Flight Hollow Stem Augers | COMPILED BY K.A.
DATUM Geodetic | DATE 2019.10.18 | LATITUDE 42.273873 | LONGITUDE -82.410747 | CHECKED BY R.N.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
178.4	Ground					
0.0	CLAYEY SILT some sand, trace gravel Stiff, Brown, Moist (FILL)	1	SS	10	178	
		2	SS	9	177	
		3	SS	11		
		4	SS	9	176	
		5	SS	6	175	1 14 39 46
174.4 4.0	CLAYEY SILT, some sand, trace gravel Stiff, Grey, Moist (TILL)	6	SS	15	174	
		7	SS	9		3 17 41 39
		8	SS	9	173	
		9	SS	10	172	
					171	
		10	SS	9	170	2 19 40 39
		11	SS	8	169	
					168	
		12	SS	9	167	
		13	SS	3	166	
			VANE		165	
		14	TW		164	2 17 38 43 P_c = 175 kPa C_c = 0.19 e_o = 0.721 SG = 2.718
163.4						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

Field vane at approx. elev. 164.8: $+^2$

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No LEB

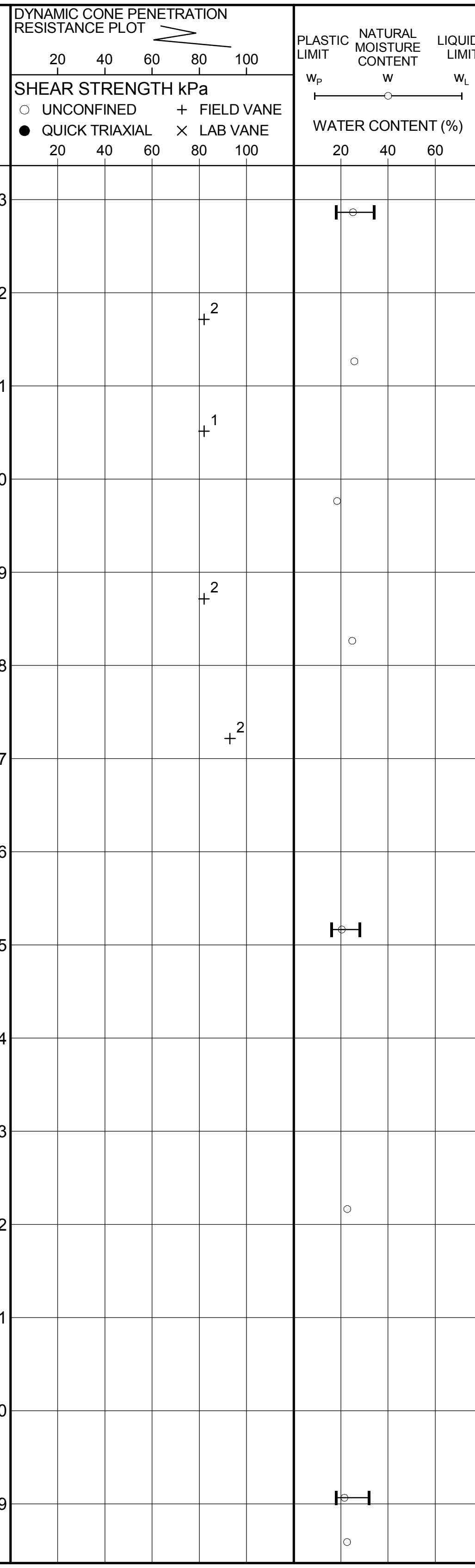
METRIC

G.W.P. 3034-19-00 LOCATION Coords: 4 681 593.3 N; 312 162.2 E ORIGINATED BY M.M.

DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.

DATUM Geodetic DATE 2019.10.18 LATITUDE 42.273873 LONGITUDE -82.410747 CHECKED BY R.N.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
163.4 / 15.0	(Cont'd) CLAYEY SILT, some sand, trace gravel. Stiff, Grey, Moist (TILL)	15	SS	3	2	19	39	40
			VANE					
		16	SS	2				
			VANE					
		17	SS	4				
			VANE					
		18	SS	5				
			VANE					
		19	SS	10	1	23	44	32
		20	SS	13				
		21	SS	6	3	20	38	39
148.4		22	SS	14				



Other column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%).

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No LEB

METRIC

G.W.P. 3034-19-00 | LOCATION Coords: 4 681 593.3 N; 312 162.2 E | ORIGINATED BY M.M.

DIST West Region HWY 401 | BOREHOLE TYPE Continuous Flight Hollow Stem Augers | COMPILED BY K.A.

DATUM Geodetic | DATE 2019.10.18 | LATITUDE 42.273873 | LONGITUDE -82.410747 | CHECKED BY R.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
148.4 30.0	End of borehole										

NOTES:
1. Groundwater was not encountered during or upon completion of drilling.
2. No cave-in was noted upon extraction of augers.

▼ Groundwater level measured in monitoring well

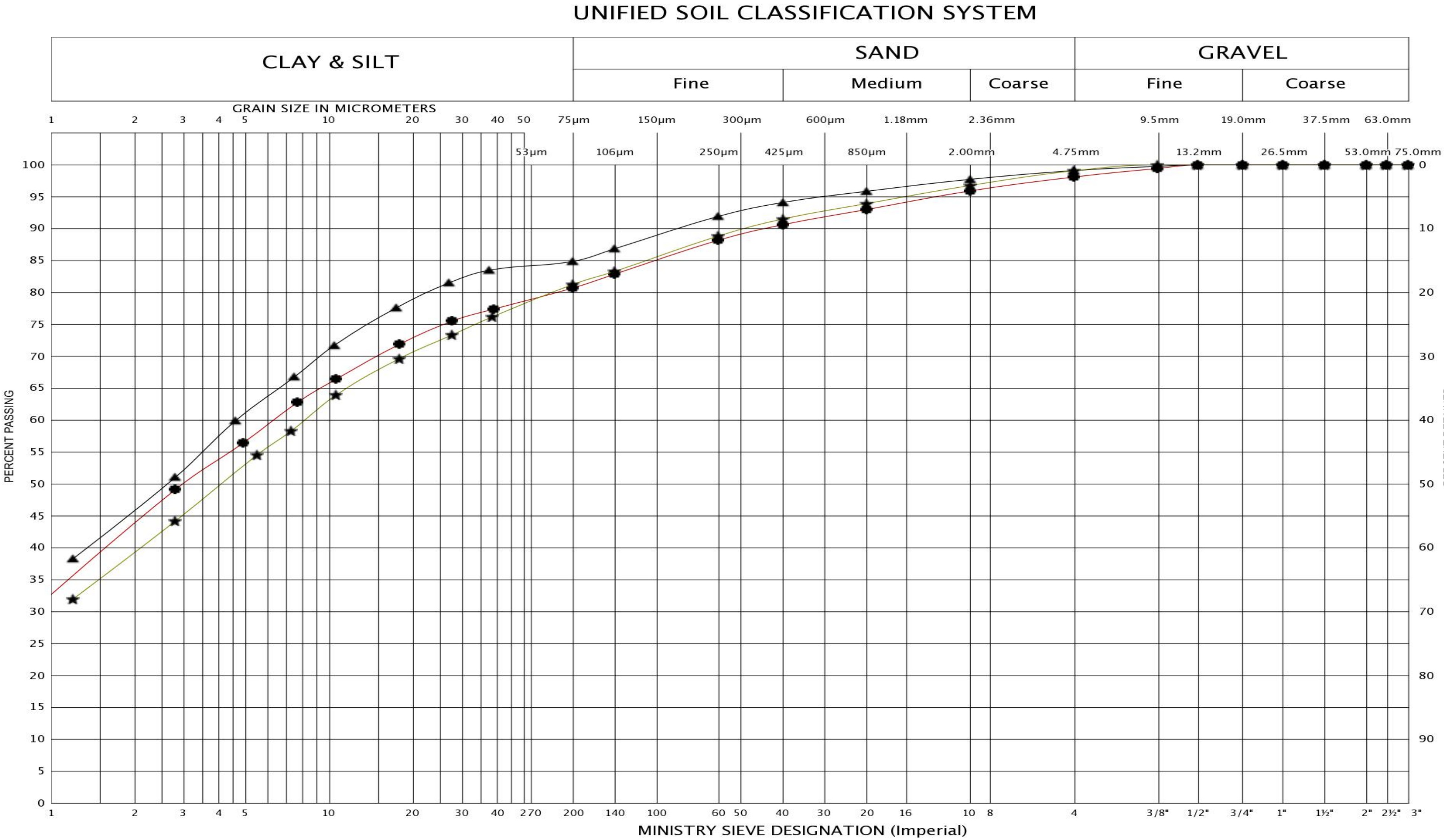
Monitoring Well Readings:

Date	Depth (m)	Elev.
Oct.24/'19	Dry	-
Oct.29/'19	7.5	170.9
Nov.07/'19	1.75	176.7

Monitoring Well Legend:
- Stick-up Monument
- Bentonite
- Filter Sand
- 19 mm PVC Screen
- Filter Bottom

$+^3, \times^3$: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

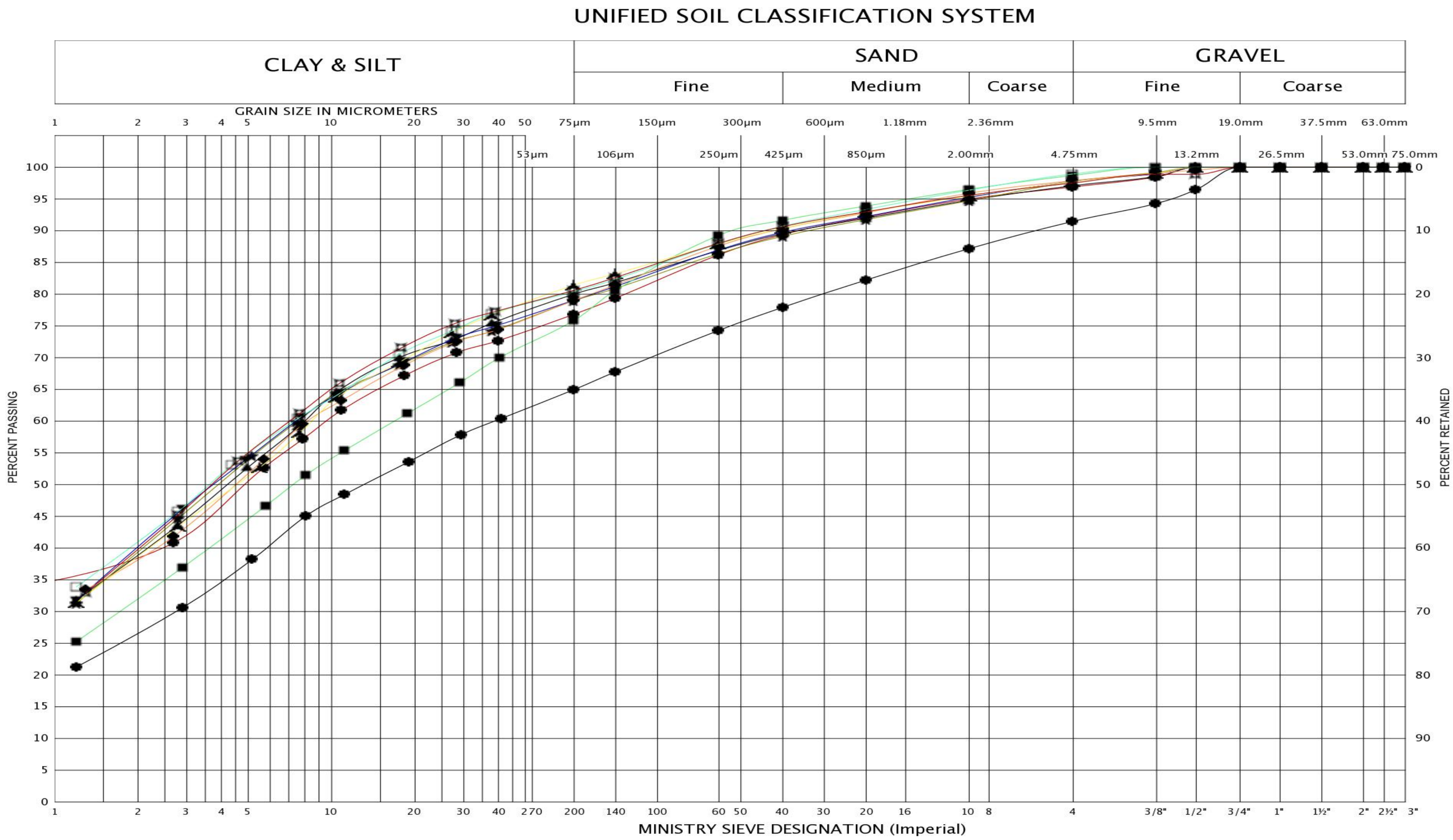


LEGEND				
	BH	LEB	LEB	LWB
	SAMPLE	14	5	6
	SYMBOL	⬢	▲	★

GRAIN SIZE DISTRIBUTION

SILTY CLAY, Some Sand, Trace Gravel (Till)

FIG No.:	GS-LC-1
HWY :	401
GWP	3034-19-00



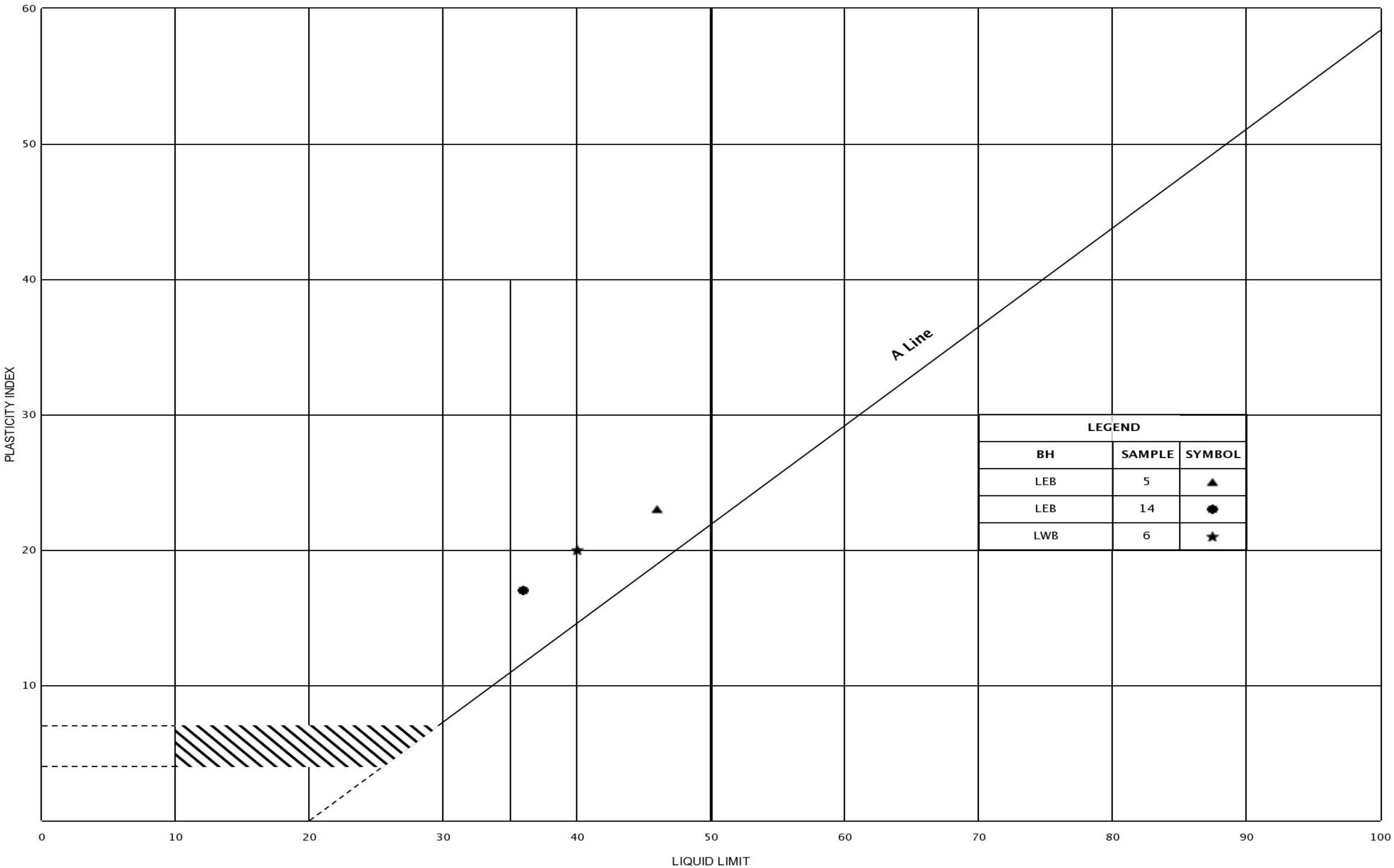
LEGEND	BH	LEB	LEB	LEB	LEB	LEB	LWB	LWB	LWB	LWB	LWB
	SAMPLE	21	7	10	15	19	21	9	13	16	19
	SYMBOL	●	▲	★	▼	■	◆	⅄	□	⌘	●

Ontario

GRAIN SIZE DISTRIBUTION

CLAYEY SILT, Some Sand to Sandy, Trace Gravel (Till)

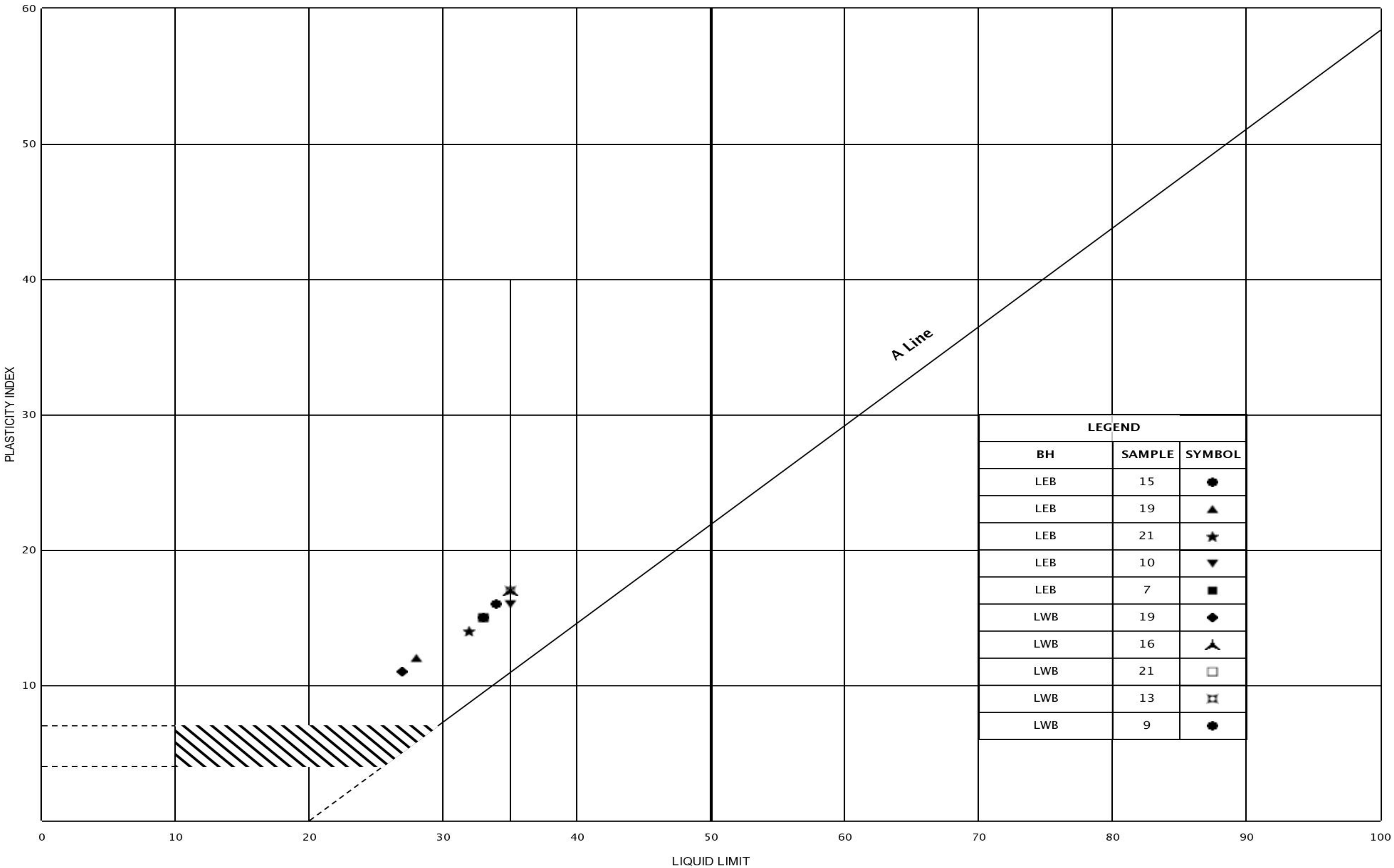
FIG No.:	GS-LC-2
HWY :	401
GWP	3034-19-00



PLASTICITY CHART

SILTY CLAY, Some Sand, Trace Gravel (Till)

FIG No.:	PC-LC-1
HWY.:	401
GWP	3034-19-00



Ontario

PLASTICITY CHART

CLAYEY SILT, Some Sand to Sandy, Trace Gravel (Till)

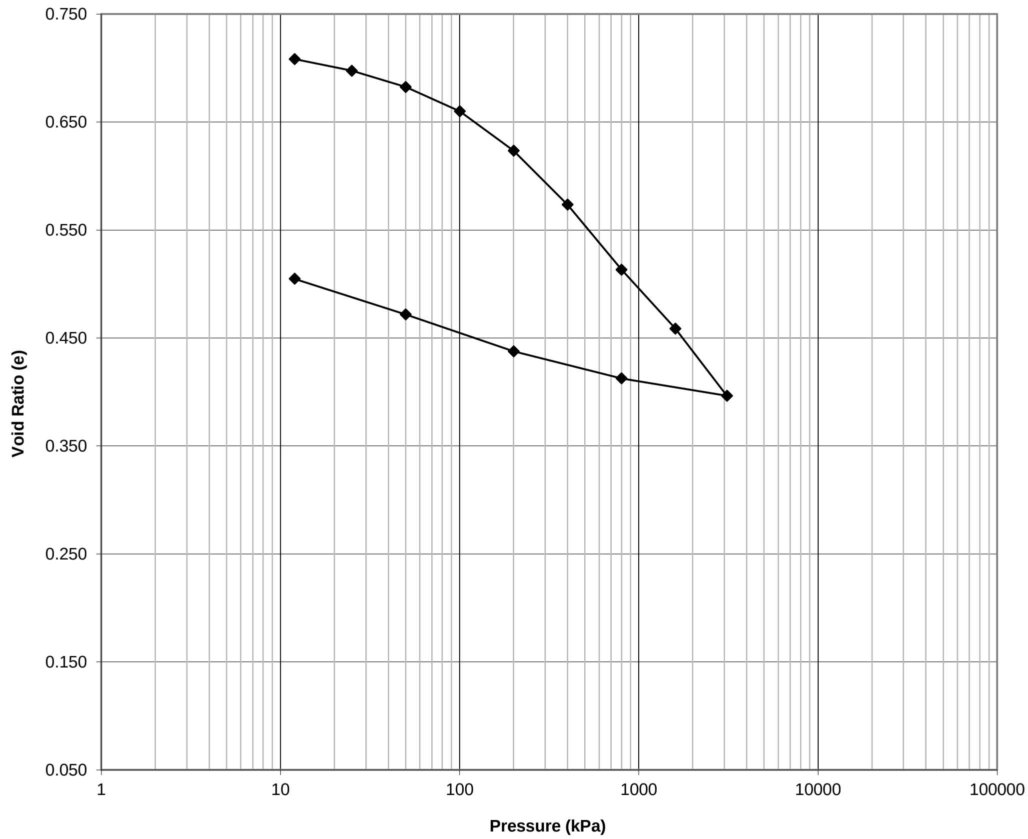
FIG No.:	PC-LC-2
HWY.:	401
GWP	3034-19-00

Peto MacCallum Ltd.
CONSULTING ENGINEERS

Consolidation Test Results
(ASTM D2435)
Highway 401,CA 3017-E-0006,Task 007 -Tilbury

Borehole LEB, Sample TW 14, Depth 14.0-14.6 m.

Void Ratio versus Log of Pressure



SOIL TYPE:	Clayey Silt Till						
e_0	=	0.721		W_L	=	36	FIGURE No: L-1
W_0	=	27.4	%	W_P	=	19	Highway 401,CA 3017-E-0006,Task 007 -Tilbury
γ	=	19.7	kN/m³	PI	=	17	
							PML Ref: 19KF030A

FINAL REPORT

CA14916-OCT19 R1

19KF030A Hwy 401 and Little Baptiste Creek

Prepared for

Peto MacCallum Ltd

FINAL REPORT

First Page

CLIENT DETAILS		LABORATORY DETAILS	
Client	Peto MacCallum Ltd	Project Specialist	Brad Moore Hon. B.Sc
		Laboratory	SGS Canada Inc.
Address	165 Cartwright Ave Toronto, ON M6A 1V5, Canada	Address	185 Concession St., Lakefield ON, K0L 2H0
Contact	Nazibur Rahman	Telephone	705-652-2143
Telephone	416-785-5110	Facsimile	705-652-6365
Facsimile	416-785-5120	Email	brad.moore@sgs.com
Email	nrahman@petomaccallum.com	SGS Reference	CA14916-OCT19
Project	19KF030A Hwy 401 and Little Baptiste Creek	Received	10/29/2019
Order Number		Approved	11/01/2019
Samples	Soil (3)	Report Number	CA14916-OCT19 R1
		Date Reported	11/01/2019

COMMENTS

Temperature of Sample upon Receipt: 3 degrees C
Cooling Agent Present:Yes
Custody Seal Present:No

Chain of Custody Number:006622

Corrosivity Index is based on the American Water Works Corrosivity Scale according to AWWA C-105. An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.

SIGNATORIES

Brad Moore Hon. B.Sc

TABLE OF CONTENTS

Client: Peto MacCallum Ltd

Project: 19KF030A Hwy 401 and Little Baptiste Creek

Project Manager: Nazibur Rahman

Samplers: K Amatya

PACKAGE: **- Corrosivity Index** (SOIL)

			Sample Number	5	6	7
			Sample Name	BH# LEB, SS/4 7.5'-9.5'	BH# LEB, SS/11 30'-32'	BH# LWB, SS/10 25'-27'
			Sample Matrix	Soil	Soil	Soil
			Sample Date	22/10/2019	22/10/2019	22/10/2019
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**
Corrosivity Index						
Corrosivity Index	none	1		6.5	4.5	4.5
Soil Redox Potential	mV	-		177	134	106
Sulphide	%	0.02		0.02	0.45	0.38
pH	pH Units	0.05		7.75	8.42	8.24
Resistivity (calculated)	ohms.cm	-9999		2260	3520	3770

PACKAGE: **- General Chemistry** (SOIL)

			Sample Number	5	6	7
			Sample Name	BH# LEB, SS/4 7.5'-9.5'	BH# LEB, SS/11 30'-32'	BH# LWB, SS/10 25'-27'
			Sample Matrix	Soil	Soil	Soil
			Sample Date	22/10/2019	22/10/2019	22/10/2019
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**
General Chemistry						
Conductivity	uS/cm	2		443	284	265

PACKAGE: **- Metals and Inorganics** (SOIL)

			Sample Number	5	6	7
			Sample Name	BH# LEB, SS/4 7.5'-9.5'	BH# LEB, SS/11 30'-32'	BH# LWB, SS/10 25'-27'
			Sample Matrix	Soil	Soil	Soil
			Sample Date	22/10/2019	22/10/2019	22/10/2019
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**
Metals and Inorganics						
Moisture Content	%	0.1		20.4	17.7	16.7

Client: Peto MacCallum Ltd

Project: 19KF030A Hwy 401 and Little Baptiste Creek

Project Manager: Nazibur Rahman

Samplers: K Amatya

PACKAGE: **- Metals and Inorganics** (SOIL)

			Sample Number	5	6	7
			Sample Name	BH# LEB, SS/4 7.5'-9.5'	BH# LEB, SS/11 30'-32'	BH# LWB, SS/10 25'-27'
			Sample Matrix	Soil	Soil	Soil
			Sample Date	22/10/2019	22/10/2019	22/10/2019
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**
Metals and Inorganics (continued)						
Sulphate	µg/g	0.4		78	110	130

PACKAGE: **- Other (ORP)** (SOIL)

			Sample Number	5	6	7
			Sample Name	BH# LEB, SS/4 7.5'-9.5'	BH# LEB, SS/11 30'-32'	BH# LWB, SS/10 25'-27'
			Sample Matrix	Soil	Soil	Soil
			Sample Date	22/10/2019	22/10/2019	22/10/2019
Parameter	**Units**	**RL**		**Result**	**Result**	**Result**
Other (ORP)						
Chloride	µg/g	0.4		170	140	45

QC SUMMARY

Anions by IC

Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-[ENV]IC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Chloride	DIO0593-OCT19	µg/g	0.4	<0.4	19	20	97	80	120	NV	75	125
Sulphate	DIO0593-OCT19	µg/g	0.4	<0.4	15	20	94	80	120	91	75	125

Carbon/Sulphur

Method: ASTM E1915-07A | Internal ref.: ME-CA-[ENV]ARD-LAK-AN-020

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Sulphide	ECS0001-NOV19	%	0.02	<0.02	5	20	114	80	120			

Conductivity

Method: SM 2510 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Conductivity	EWL0553-OCT19	uS/cm	2	< 0.002	0	10	100	90	110	NA		

QC SUMMARY

pH

Method: SM 4500 I Internal ref.: ME-CA-[ENV]EWL-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
pH	EWL0553-OCT19	pH Units	0.05	NA	0		100			NA		

Method Blank: a blank matrix that is carried through the entire analytical procedure. Used to assess laboratory contamination.

Duplicate: Paired analysis of a separate portion of the same sample that is carried through the entire analytical procedure. Used to evaluate measurement precision.

LCS/Spike Blank: Laboratory control sample or spike blank refer to a blank matrix to which a known amount of analyte has been added. Used to evaluate analyte recovery and laboratory accuracy without sample matrix effects.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate laboratory accuracy with sample matrix effects.

Reference Material: a material or substance matrix matched to the samples that contains a known amount of the analyte of interest. A reference material may be used in place of a matrix spike.

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

Multielement Scan Qualifier: as the number of analytes in a scan increases, so does the chance of a limit exceedance by random chance as opposed to a real method problem. Thus, in multielement scans, for the LCS and matrix spike, up to 10% of the analytes may exceed the quoted limits by up to 10% absolute and the spike is considered acceptable.

Duplicate Qualifier: for duplicates as the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Matrix Spike Qualifier: for matrix spikes, as the concentration of the native analyte increases, the uncertainty of the matrix spike recovery increases. Thus, the matrix spike acceptance limits apply only when the concentration of the matrix spike is greater than or equal to the concentration of the native analyte.

LEGEND

FOOTNOTES

NSS Insufficient sample for analysis.
RL Reporting Limit.
↑ Reporting limit raised.
↓ Reporting limit lowered.
NA The sample was not analysed for this analyte
ND Non Detect

Samples analysed as received. Solid samples expressed on a dry weight basis. “Temperature Upon Receipt” is representative of the whole shipment and may not reflect the temperature of individual samples.

Analysis conducted on samples submitted pursuant to or as part of Reg. 153/04, are in accordance to the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act” published by the Ministry and dated March 9, 2004 as amended.

-- End of Analytical Report --

APPENDIX B

Previous Borehole Logs and Drawings (GEOCRES No. 40J08-017)

DEPARTMENT OF HIGHWAYS OF ONTARIO

SOILS REPORT

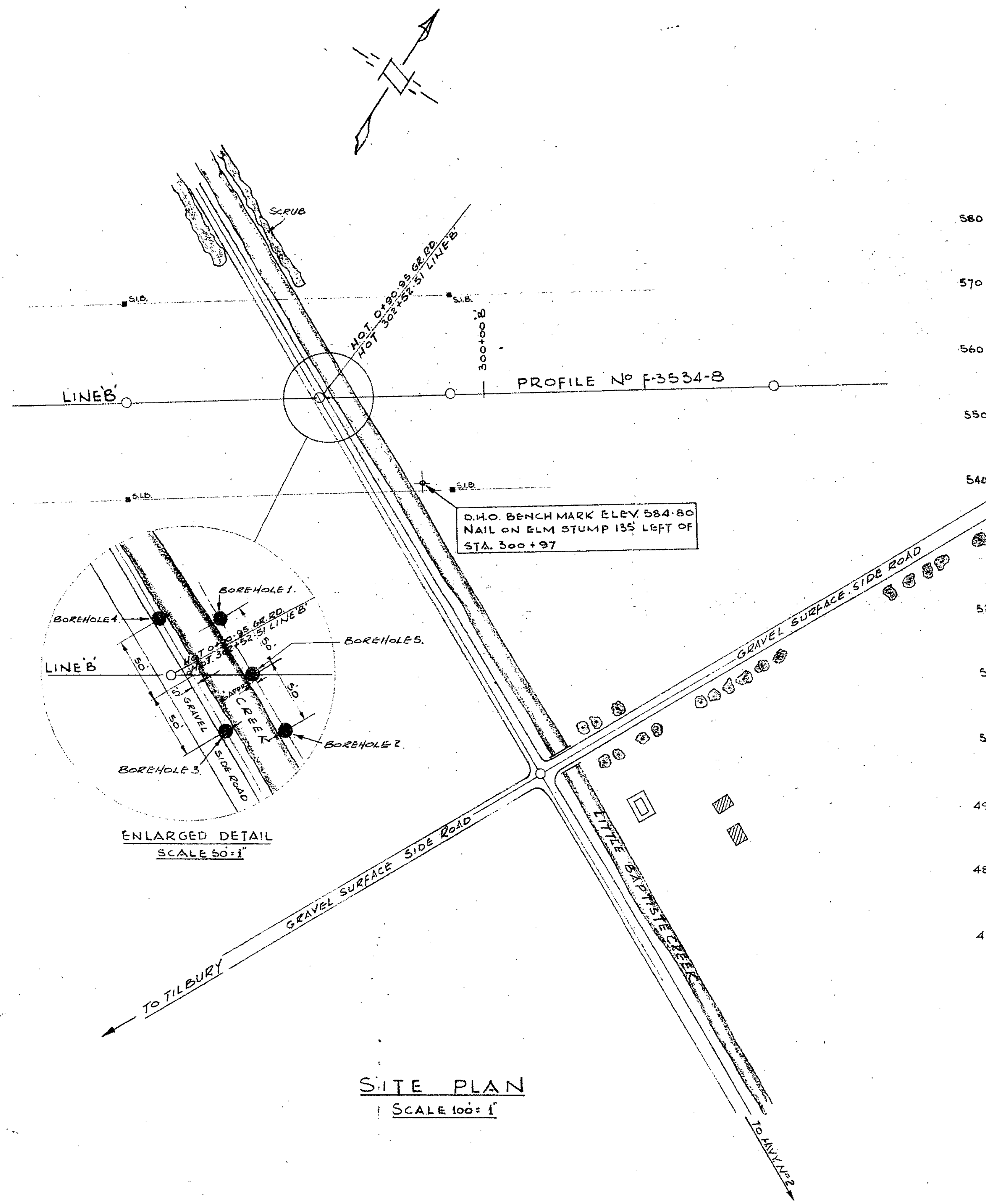
for

HWY. 401 - LITTLE BAPTISTE CREEK CROSSING

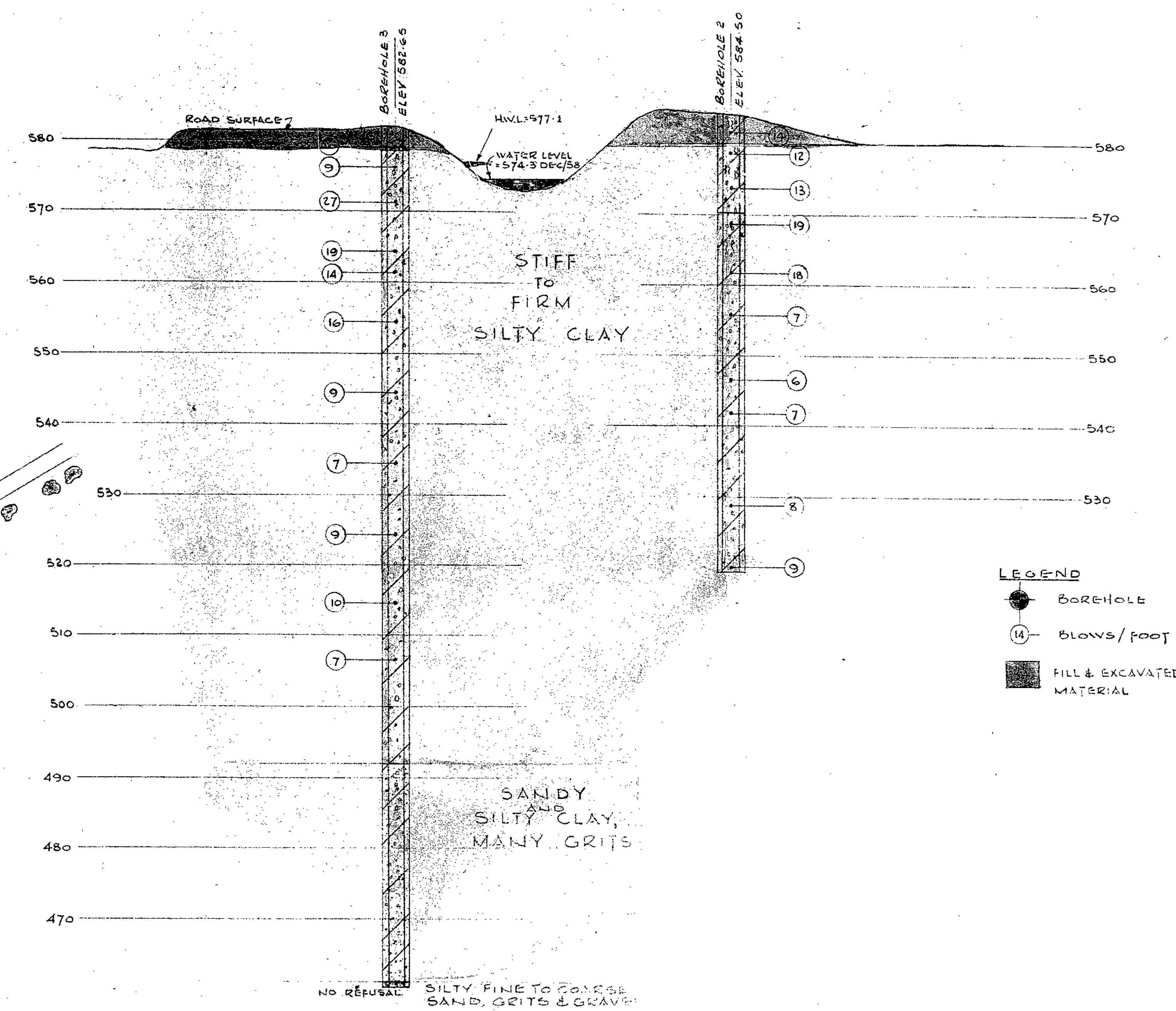
W.P. 162 - 58

January, 1959

e. m. peto associates ltd.



SITE PLAN
SCALE 100':1"



SECTION THROUGH HOLES 3 & 2

PROFILE SCALES VERT 10':1"
HOR. 10':1"

NOTE: SEE BOREHOLE LOGS FOR COMPLETE SOIL DETAILS

REGISTERED PROFESSIONAL ENGINEER
E. M. PETO
PROVINCE OF ONTARIO

e.m. peto & associates ltd.

SOIL SITE INVESTIGATION
AT
HWY. 401 - LITTLE BAPTISTE CR. CROSSING
TILBURY ONTARIO
FOR
DEPT. of HIGHWAYS of ONTARIO

OUR JOB No. 58162 DATE 15 JAN/59
CLIENTS PLAN No. E-3413-1 & F3534-8 C J.W.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401-Little Baptiste Cr. Crossing Job No. 58162 Borehole No. 1

Client Dept. of Highways of Ontario. Casing BX ($2\frac{1}{2}$" diam.) Boring Date Dec. 16-18, 1958.

Datum Geodetic Compiled By M. Mindess Checked By E. M. Peto

SAMPLE CONDITION

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

Soil Description	Colour	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	Water Levels, Soil Moisture & Remarks
			0' 0" 584.3					
SANDY CLAY SILTY CLAY, FISSURED, ORGANIC TRACES. AS ABOVE	MOTTLED BROWN MOTTLED BROWN MOTTLED	STIFF	5' 0"		1	S.S.	13	NAT. M.C.=16.4% DRIER THAN PLASTIC LIMIT.
SILTY CLAY, FISSURED.	GREY-BROWN	STIFF	10' 0"		2	S.S.	14	NAT. M.C.=21.9% DRIER THAN PLASTIC LIMIT.
SILTY CLAY, GRITS AND STONES TO $\frac{3}{4}$" SIZE.	BROWN	STIFF	14' 0" 570.3		3	S.S.	19	NAT. M.C.=17.6% AT PLASTIC LIMIT.
SILTY CLAY, MANY BLACK GRITS.	DARK GREY	STIFF	20' 0"		4	S.S.	22	NAT. M.C.=18.1% WETTER THAN PLASTIC LIMIT.
STRATIFIED SILTY CLAY, NUMEROUS BLACK GRITS.	DARK GREY	FIRM TO STIFF	25' 0"		5	S.S.	11	NAT. M.C.=20.4% MUCH WETTER THAN PLASTIC LIMIT
SILTY CLAY, GRITS	DARK GREY	FIRM			6 A B C	2" S.L.	PUSHED	NAT. M.C.=22.1% C=1370 P.S.F. @ 20% STRAIN
SILTY CLAY, GRITS AND LIMESTONE PEBBLES TO $\frac{1}{2}$" SIZE	DARK GREY	FIRM	30' 0"		7	S.S.	12	MUCH WETTER THAN PLASTIC LIMIT
SILTY CLAY, GRITS	GREY	FIRM	35' 0"		8	2" S.L.	PUSHED	NAT. M.C.=23.3–25.3% C= 1150 — 793 P.S.F. @ 20% STRAIN.
SILTY CLAY, BLACK GRITS	GREY	FIRM	40' 0"		9	S.S.	8	MUCH WETTER THAN PLASTIC LIMIT
AS ABOVE	"	SOFT			10 A B C	2" S.L.	PUSHED	NAT. M.C.=25.1% C=650 P.S.F.
AS ABOVE	GREY	SOFT	45' 0"		11	S.S.	7	MUCH WETTER THAN PLASTIC LIMIT
		"	50' 0"		12	2" S.L.	PUSHED	
AS ABOVE	GREY	SOFT	55' 0"		13	S.S.	8	AS ABOVE
AS ABOVE	GREY	SOFT	60' 0"		14	2" S.L.	PUSHED	NAT. M.C.=26.6–26.3% C= 416 P.S.F.
AS ABOVE, 3" SEAM OF GRITS AND PEA GRAVEL IN MATRIX OF SILTY CLAY.	"	"	65' 0"		15	2" S.L.	PUSHED	NAT. M.C.=22.6% C=650 P.S.F.
			70' 0"		16	S.S.	8	
					17	2" S.L.	PUSHED	
SILTY CLAY, BLACK GRITS.	GREY	SOFT	75' 0"		18	S.S.	7	MUCH WETTER THAN PLASTIC LIMIT. C= APPROX. 300 P.S.F.
			80' 0"					
AS ABOVE	GREY	SOFT	85' 0"		19	S.S.	9	AS ABOVE
AS ABOVE			90' 0"			WASHED OPEN-END A-ROD		CASING WITHDRAWN DEC 18 HOLE CAVED IN TO 35' DEPTH DEPTH TO WATER=11'0" DEC. 20,1958
AS ABOVE	GREY	SOFT	95' 0" 489.3		20	DRIVE A-ROD	(6)	MUCH WETTER THAN PLASTIC LIMIT

HOLE TERMINATED. NO STIFFENING OR REFUSAL

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401-Little Baptiste Cr. Job No. 58162 Borehole No. 2

Client Dept. of Highways of Ontario Casing 4" pipe + BX Boring Date Dec. 18-20, 1958.

Datum Geodetic Compiled By M. Mindess Checked By E. M. Peto

SAMPLE CONDITION

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft.	WATER LEVELS, SOIL MOISTURE & REMARKS
GROUND FROZEN TO 1'3".			0' 0" 584.5					
FISSURED SILTY CLAY, GRITS, ORGANIC TRACES.	GREY-BROWN				1A	SAMPLE FROM CASING		AT PLASTIC LIMIT.
SILTY CLAY, GRITS AND PEBBLES, SILT POCKETS.	MOTTLED GREY-BROWN	STIFF	5' 0"		1B	S.S.	14	NAT. M.C.=20.5 % DRIER THAN PLASTIC LIMIT.
SILTY CLAY, GRITS AND PEBBLES, SOME ORGANIC MATTER	AS ABOVE	STIFF	10' 0"		2	S.S.	12	NAT. M.C.=19.8 % DRIER THAN PLASTIC LIMIT.
					2A	SAMPLE FROM CASING	–	
SILTY CLAY, GRITS AND PEBBLES.	AS ABOVE	STIFF	14' 0" 570.5		3	S.S.	13	NAT. M.C.=23.7 % DRIER THAN PLASTIC LIMIT.
STRATIFIED SILTY CLAY, BLACK GRITS.	DARK GREY	STIFF	20' 0"		4	S.S.	19	NAT. M.C.= 18.7 % WETTER THAN PLASTIC LIMIT. S.G.=2.69
		"			5 A B C	3" S.L.	TAPPED	
		STIFF	25' 0"		6	S.S.	18	
			30' 0"		7	3" S.L.	PUSHED	
SILTY CLAY, GRITS.	GREY	FIRM	35' 0"		8	S.S.	7	MUCH WETTER THAN PLASTIC LIMIT.
					9	3" S.L.	PUSHED	
AS ABOVE	GREY	FIRM.	40' 0"		10	S.S.	6	MUCH WETTER THAN PLASTIC LIMIT.
					11 A B C	2" S.L.	PUSHED	
AS ABOVE	GREY	FIRM.	45' 0"		12	S.S.	7	AS ABOVE
			50' 0"					
" "			55' 0"		13 A B	2" S.L.	PUSHED	
AS ABOVE	GREY	FIRM	60' 0"		14	S.S.	8	MUCH WETTER THAN PLASTIC LIMIT.
					15 A B	2" S.L.	PUSHED	
SILTY CLAY, BLACK GRITS.	GREY	FIRM	65' 0" 519.5		16	S.S.	9	MUCH WETTER THAN PLASTIC LIMIT.

HOLE TERMINATED.
NO STIFFENING OR REFUSAL

CASING WITHDRAWN, DEC. 20, 58
DEPTH TO WATER= 12'6"
DEC. 22, '58.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401-Little Baptiste Cr. Crossing Job No. 58162 Borehole No. 3

Client Dept. of Highways of Ontario. Casing BX ($2\frac{1}{2}$" diam.) Boring Date Dec. 22-Jan. 1, 1958-9.

Datum Geodetic Compiled By M. Mindess Checked By E. M. Peto

SAMPLE CONDITION

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN-SITU VANE SHEAR TEST
- Qu UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

Soil Description	Colour	Density or Consistency	Depth Elevation	Sample No. and Condition	Sample Type	No. of Blows per ft.	Water Levels, Soil Moisture & Remarks
			0' 0" 582.65				
SILTY CLAY, GRITS.	MOTTLED GREY-BROWN.			1A	SAMPLE FROM CASING		AT PLASTIC LIMIT
AS ABOVE, ORGANIC CONTENT. ORGANIC SILTY LOAM.	DARK BROWN.	STIFF	3' 0" / 3'6" 579.6	1B	S.S.	10	NAT. M.C.=29.9 %. WETTER THAN PLASTIC LIMIT.
	MOTTLED		5' 0"				
SILTY CLAY, GRITS.	GREY-BROWN	STIFF		2	S.S.	9	NAT. M.C.=22.4 %. WETTER THAN PLASTIC LIMIT. C = APPROX. 2300 PSF
			10' 0"				
STRATIFIED SILTY CLAY, BLACK GRITS.	BROWN	STIFF		3	S.S.	27	NAT. M.C.=18.2 %. WETTER THAN PLASTIC LIMIT.
			15' 0"				
				4 A B C	2" S.L.	PUSHED & TAPPED	
AS ABOVE.	DARK GREY	STIFF		5	S.S.	19	MUCH WETTER THAN PLASTIC LIMIT.
			20' 0"				
AS ABOVE.	GREY	STIFF		6	S.S.	14	AS ABOVE.
			25' 0"				
				7 A B C	2" S.L.	PUSHED.	
AS ABOVE.	GREY	STIFF		8	S.S.	16	MUCH WETTER THAN PLASTIC LIMIT.
			30' 0"				
			35' 0"				
				9 A B C	S.L.	PUSHED	
SILTY CLAY, GRITS TO $\frac{1}{2}$" SIZE.	GREY	FIRM		10	S.S.	9	MUCH WETTER THAN PLASTIC LIMIT.
			40' 0"				
			45' 0"				
				11 A B C	2" S.L.	PUSHED.	
SILTY CLAY, GRITS.	GREY	FIRM.		12	S.S.	7	MUCH WETTER THAN PLASTIC LIMIT.
			50' 0"				
			55' 0"				
AS ABOVE	GREY	FIRM.		13 A B C	2" S.L.	PUSHED	
AS ABOVE	"	"		14	S.S.	9	AS ABOVE.
			60' 0"				
			65' 0"				
"	"	"		15 A B C	2" S.L.	PUSHED	
SILTY CLAY, GRITS.	GREY	FIRM		16	S.S.	10	MUCH WETTER THAN PLASTIC LIMIT.
			70' 0"				
AS ABOVE	"	"		17 A B C	2" S.L.	PUSHED	
			75' 0"				
SILTY CLAY, NUMEROUS GRITS.	GREY	FIRM		18	S.S.	7	AS ABOVE.
				WASHED A-ROD AHEAD TO PROBE FOR REFUSAL. NO CASING DRIVEN BEYOND 75'.			
			80' 0"				
			85' 0"				
AS ABOVE	"	"					
			90' 0" 492.6				
GRADING TO SANDY AND SILTY CLAY, MANY GRITS.	GREY	PROBABLY FIRM					
			95' 0"				
			100' 0"				
			105' 0"				
			110' 0"				
			115' 0"				CASING WITHDRAWN JAN. 1, 1959. HOLE CAVED IN TO 37' DEPTH. NO WATER, JAN. 3, 1959
AS ABOVE	"	"	121' 0"				
MEDIUM TO COARSE SAND, GRITS AND LIMESTONE FRAGMENTS, IN MATRIX OF CLAYEY SILT.	GREY	VERY DENSE	122' 0" 460.6	19	DROVE OPEN-END A-ROD	(68)	SATURATED

HOLE TERMINATED. NO REFUSAL.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401-Little Baptiste Cr. Crossing Job No. 58162 Borehole No. 4

Client Dept. of Highways of Ontario Casing 4" pipe + BX Boring Date Jan. 2-5, 1959.

Datum Geodetic Compiled By M. Mindess Checked By E. M. Peto

SAMPLE CONDITION	SAMPLE TYPE	ABBREVIATIONS
UNDISTURBED	S.S. 2" STANDARD SPLIT TUBE SAMPLE	V.T. IN SITU VANE SHEAR TEST
FAIR	S.L. SPLIT BARREL WITH LINERS	Q/u UNCONFINED COMPRESSIVE STRENGTH
DISTURBED	S.T. THIN-WALLED SHELBY TUBE SAMPLE	W.L. WATER LEVEL IN CASING
LOST	W.S. WASH SAMPLE	W.T. GROUND WATER TABLE IN SOIL
	R.C. ROCK CORE	

SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
DEPTH OF FROST = 24"			0' 0"					
SILTY CLAY, GRITS AND PEBBLES.	GREY-BROWN.		582.1			SAMPLE FROM CASING		AT PLASTIC LIMIT.
AS ABOVE, ORGANIC CONTENT.	MOTTLED GREY-BROWN	FIRM			1	S.S.	6	NAT. M.C.=24.2% AT PLASTIC LIMIT.
			5' 0"					
SILTY CLAY, GRITS AND PEBBLES.	AS ABOVE	STIFF			2	S.S.	10	NAT. M.C.=24.3% WETTER THAN PLASTIC LIMIT. C=2000 P.S.F.
			10' 0"					
STRATIFIED SILTY CLAY, GRITS AND ROCK FRAGMENTS TO 2" SIZE.	BROWN	STIFF			3	S.S.	14	NAT. M.C.=17.8% DRIER THAN PLASTIC LIMIT.
			15' 0"					
STRATIFIED SILTY CLAY, GRITS.	GREY	STIFF			4	S.S.	12	NAT. M.C.=18.4% WETTER THAN PLASTIC LIMIT.
			20' 0"					
					5	3" S.L.	TAPPED	
					6	3" S.T.	TAPPED	
AS ABOVE	GREY	FIRM TO STIFF	25' 0"		7	S.S.	11	WETTER THAN PLASTIC LIMIT. NAT. M.C.= 20.6%
SMALL BOULDER AT 26'								
			30' 0"					
SILTY CLAY, GRITS	GREY	FIRM			8	S.S.	8	MUCH WETTER THAN PLASTIC LIMIT.
					9	3" S.L.	PUSHED	
			35' 0"					
AS ABOVE	"	"			10	3" S.T.	PUSHED.	
			40' 0"					
AS ABOVE	"	"			11	S.S.	8	NAT. M.C.=26.5% MUCH WETTER THAN PLASTIC LIMIT.
			45' 0"					
					12	3" S.T.	PUSHED	
AS ABOVE	"	"			13	S.S.	9	MUCH WETTER THAN PLASTIC LIMIT.
			50' 0"					
			55' 0"					
AS ABOVE	GREY	FIRM			14	S.S.	7	AS ABOVE
			60' 0"					
					15	S.S.	8	NAT. M.C.=24.9% MUCH WETTER THAN PLASTIC LIMIT.
			65' 0"					
					16	2" S.L.	PUSHED	
SILTY CLAY, GRITS	GREY	FIRM	67' 0" 515.1		17	S.S.	10	MUCH WETTER THAN PLASTIC LIMIT.

HOLE TERMINATED. NO STIFFENING OR REFUSAL.

CASING WITHDRAWN, JAN. 5, 1959.
HOLE CAVED IN TO 34' DEPTH.
DEPTH TO WATER = 29', JAN. 6.

e. m. peto associates ltd.

SOIL ENGINEERING SERVICE – TORONTO, ONTARIO

BOREHOLE LOG

Job Name Hwy. 401-Little Baptiste Cr. Crossing Job No. 58162 Borehole No. 5

Client Dept. of Highways of Ontario. Casing BX ($2\frac{1}{2}$" diam.) Boring Date Jan. 15, 1959.

Datum Geodetic Compiled By M. Mindess Checked By E. M. Peto

SAMPLE CONDITION

- UNDISTURBED
- FAIR
- DISTURBED
- LOST

SAMPLE TYPE

- S.S. 2" STANDARD SPLIT TUBE SAMPLE
- S.L. SPLIT BARREL WITH LINERS
- S.T. THIN-WALLED SHELBY TUBE SAMPLE
- W.S. WASH SAMPLE
- R.C. ROCK CORE

ABBREVIATIONS

- V.T. IN SITU VANE SHEAR TEST
- Q/u UNCONFINED COMPRESSIVE STRENGTH
- W.L. WATER LEVEL IN CASING
- W.T. GROUND WATER TABLE IN SOIL

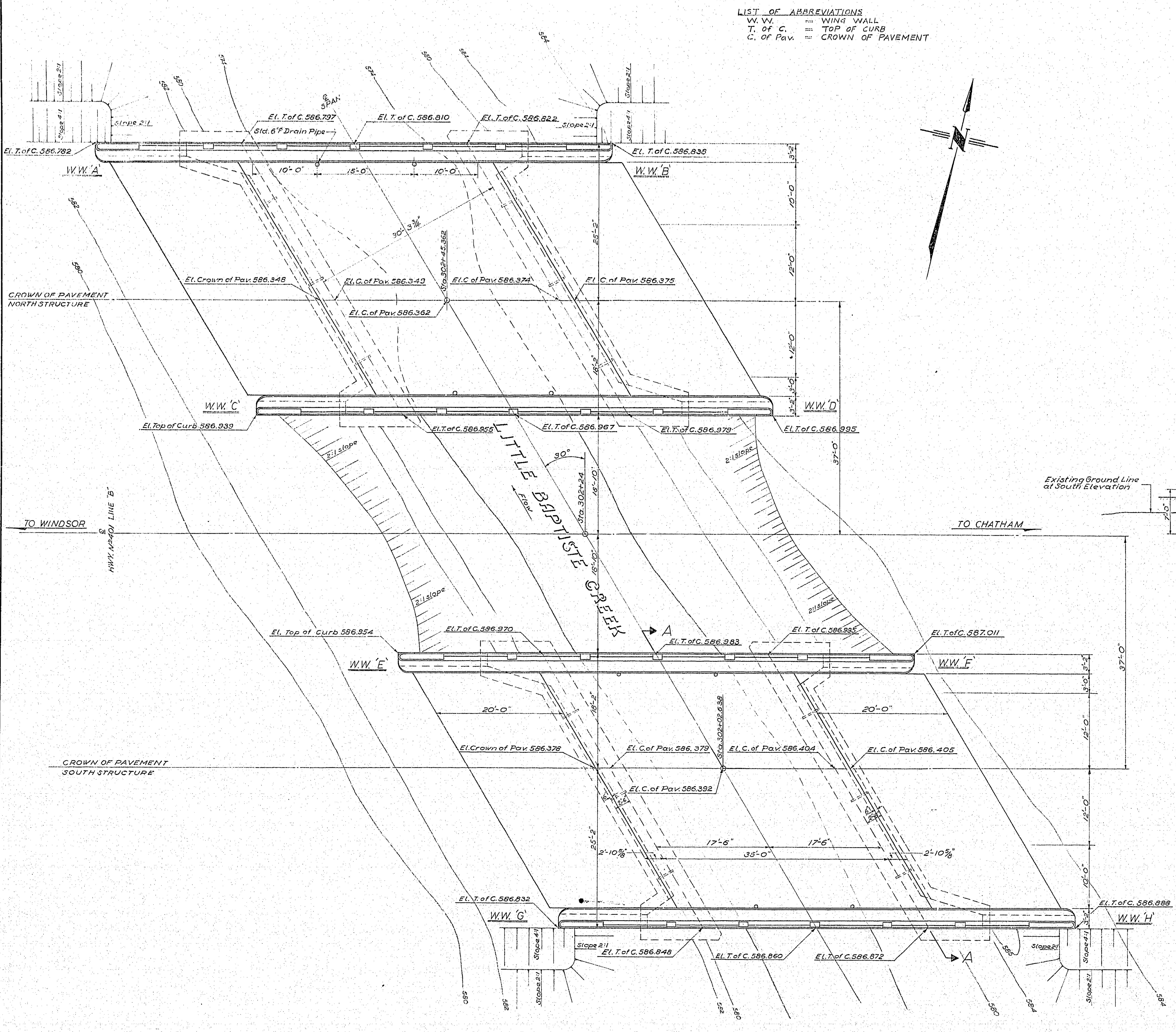
SOIL DESCRIPTION	COLOUR	Density or Consistency	Depth Elevation	Legend	Sample No. and Condition	Sample Type	No. of Blows per Ft	WATER LEVELS, SOIL MOISTURE & REMARKS
EXCAVATED MATERIAL: FISSURED SILTY AND SANDY CLAY, GRITS AND PEBBLES, ORGANIC TRACES	MOTTLED GREY-BROWN		0' 0" 584.4					
ORGANIC SILTY CLAY TOPSOIL. SILTY CLAY, GRITS.	DARK BROWN MOTTLED GREY-BROWN.	VERY STIFF	5' 0"		1 A B C	2" S.L.	TAPPED	DRIER THAN PLASTIC LIMIT NAT. M.C. = 21.8 %.
SILTY CLAY, GRITS.	MOTTLED GREY-BROWN.	VERY STIFF	10' 0"		2 A B C	2" S.L.	TAPPED	WETTER THAN PLASTIC LIMIT NAT. M.C. = 19.2 %.
SILTY CLAY, GRITS AND PEBBLES.	GREY	STIFF	14' 0" 15' 0"		3 A B C	2" S.L.	TAPPED	WETTER THAN PLASTIC LIMIT. NAT. M.C. = 18.2 %.
AS ABOVE	"	FIRM	20' 0"		4 A B C	2" S.L.	TAPPED	WETTER THAN PLASTIC LIMIT. NAT. M.C. = 19.7 %.
AS ABOVE	GREY	FIRM	25' 0" 26' 6" 557.9		5 A B C	2" S.L.	PUSHED	MUCH WETTER THAN PLASTIC LIMIT. NAT. M.C. = 20.1 % TO 19.7 %.

HOLE TERMINATED.

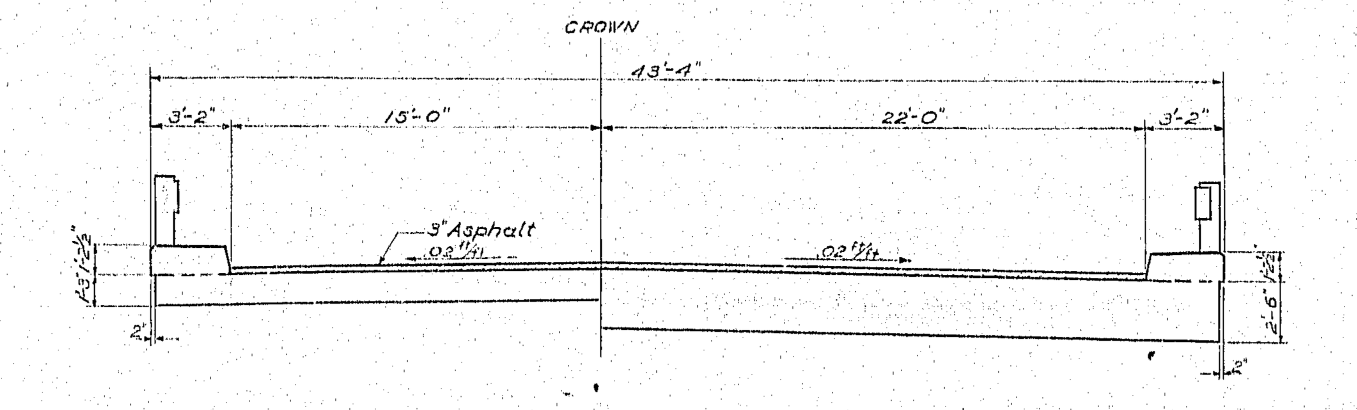
HOLE TO 25 FT. DEPTH, ONLY UPPER 10 FT. CASED, NO WATER OVERNIGHT JAN. 15 TO JAN. 16, 1959.

LIST OF ABBREVIATIONS

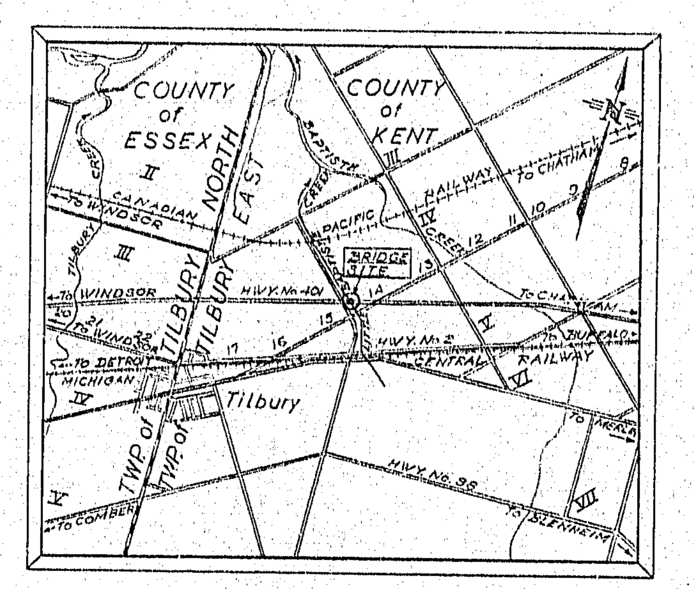
W. W.	=	WING WALL
T. of C.	=	TOP OF CURB
C. of Pav.	=	CROWN OF PAVEMENT



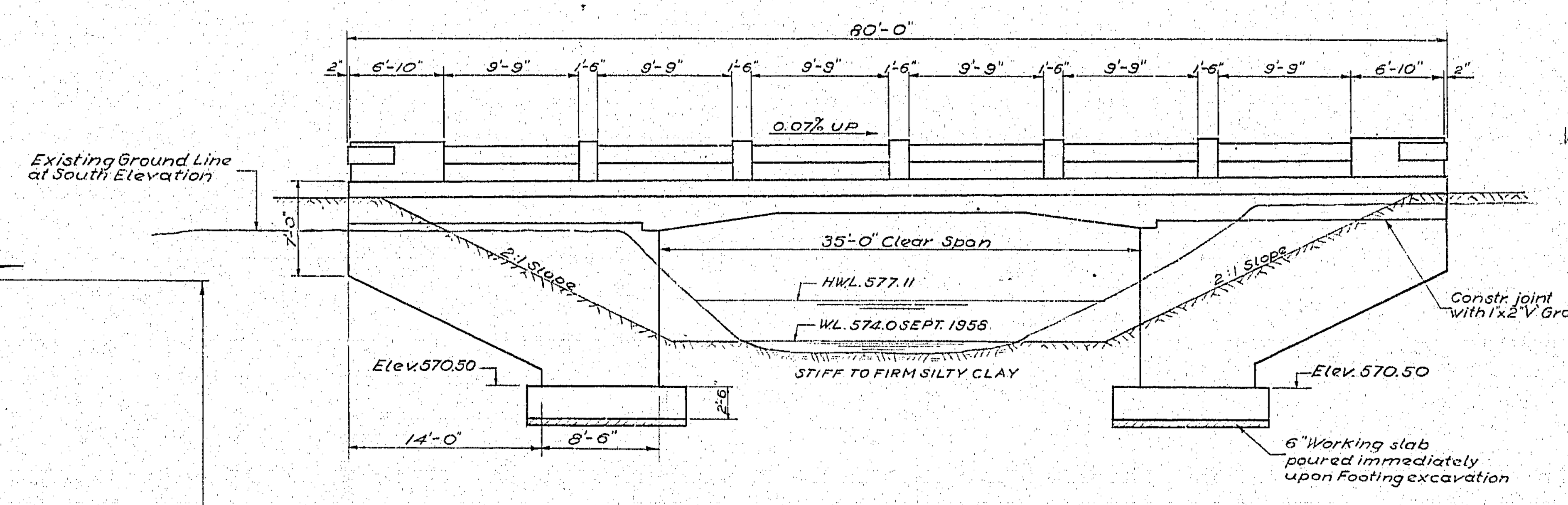
PLAN
Scale: 1/8 in.=1 ft.



SECTION A-A
Scale: 3/16 in.=1 ft.



KEY PLAN
SCALE: 1 IN=2 MI.



SOUTH ELEVATION
Scale: 1/8 in.=1 ft.

SKEW=30°
sin. 30° = 0.50000
cos. 30° = 0.86603
tan. 30° = 0.57735
cot. 30° = 1.73205
sec. 30° = 1.15470
cosec. 30° = 2.00000

NOTE TO DISTRICT ENGINEERS

Concrete work on this structure must not be commenced until monuments to fix control points have been erected and checked by the District Engineer.

NOTE TO CONTRACTOR

Structure to be built in accordance with Form Nº9 and the Special Provisions, extra copies of which may be obtained from the District Engineer.

All Construction Joints must be approved by the Bridge Engineer.

CONCRETE MIX

Minimum Concrete Strength at 28 Days

Working Slab & Footings	— 2500 p.s.i. - Max Size Aggregate	1½ in
Structure	— 3000 p.s.i. - " " "	¾ in.
Above curbs	— 3000 p.s.i. - " " "	¾ in.
Approach Slab	— 3000 p.s.i. - " " "	¾ in.

An approved admixture supplied by the Department will be added to all concrete as specified by the Engineer.

BORING DATA

The complete soil investigation report BA. 561 . . may be examined at the Bridge Office, 280 Davenport Rd. Toronto. The Department does not guarantee the accuracy of this report or the abridged version shown on these plans.

REINFORCING STEEL

Clear Cover in Footings 3 in.
" " " Structure 2 in.
" " above Curbs 1 in.
" " " Approach Slab 2 in.
} or as specified

CONSTRUCTION NOTES

All exposed edges to be chamfered 1 in. unless otherwise noted.

PRINT RECORD		
NO.	FOR	DATE
30	TENDER	3.11.59
10	DuCu F2	20.1.60
30	TENDER	8.3.61
10	DuCu F2	25.5.61

LIST OF DRAWINGS

D-4382-1 GENERAL LAYOUT
D-4382-2 DETAILS OF FOOTINGS, ABUTMENTS & DECK
D-4382-3 DETAILS OF WINGWALLS, APPROACH SLAB, CURB, HANDRAIL & POSTS
D-4382-4 REINFORCING STEEL SCHEDULE
D-4382-5 REINFORCING STEEL SCHEDULE

W.P. 162-58

DEPARTMENT OF HIGHWAYS: -ONTARIO-
BRIDGE OFFICE: -TORONTO

LITTLE BAPTISTE CREEK BRIDGE

THE KING'S HIGHWAY No. 401 DIST. No. 1
CO. OF KENT
TWP. OF TILBURY EAST LOT 14 & 15 CON. IV

GENERAL LAYOUT

APPROVED
BRIDGE ENGINEER
DESIGN ENGINEER

DESIGN	P.O.L.	CHECK	B.F.	CONTRACT NUMBERS	CATV ELLEY 59-304	APPROACH SLAB 61-109	GENERAL 59-332
DRAWING	P. Sch.	CHECK	B.F.	LOADING			
TRACING		CHECK		H_{20}-S_{16}	DRAWING NUMBER ~~D-4382-1~~		
DATE JULY 1959							

REFERENCE PLANS
Prof. NºF-3534-8
Plan NºF-3534-7
E-3513-1

REVISIONS: DATE BY DESCRIPTION

TWP 104-187-1-B

~~13-187~~ ~~D4382 1 to 5~~

APPENDIX C

List of Standard Specifications Relevant to Report

LIST OF STANDARD SPECIFICATIONS RELEVANT TO REPORT

DOCUMENT	TITLE
OPSS.PROV 405	Construction Specification for Pipe Subdrains
OPSS 501	Construction Specification for Compacting
OPSS.PROV 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.PROV 539	Temporary Protection Systems
OPSS 902	Excavation and Backfilling of Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
OPSD 3090.101	Foundation, Frost Penetration depths for Southern Ontario
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement
OPSD 3121.150	Wall, Retaining, Backfill, Minimum Granular Requirement
SP 105S09	Amendment to OPSS 539, November 2014
SP 109F57	Amendment to OPSS 903, April 2016
SP 109S12	Amendment to OPSS 902, November 2010
NSSP FOUN0003	Dewatering Structure Excavations, Amendment to OPSS 902